American Association of State Highway and Transportation Officials
AASHTO

Carlos Braceras, P.E., President
Executive Director, Utah Department of Transportation

Jim Tymon, Executive Director

444 North Capitol Street NW, Suite 249, Washington, DC 20001
(202) 624-5800 • Fax: (202) 624-5806 • www.transportation.org

ERRATA for

A Policy on Geometric Design of Highways and Streets

October 2019

Dear Customer:

Recently, we were made aware of some technical revisions that need to be applied to the 2018 *A Policy on Geometric Design of Highways and Streets*, 7th Edition.

Please replace the existing text with the corrected text to ensure that your edition is both accurate and current. Additional copies of this erratum can be downloaded from AASHTO's online bookstore at

http://downloads.transportation.org/GDHS-7-Errata.pdf

AASHTO staff sincerely apologizes for any inconvenience to our readers.

Page	Existing	Corrected
Chapter 2		
2-52	In Section 2.6.3, paragraph 1, the reference callout is incorrect.	In Section 2.6.3, paragraph 1, changed the reference callout from "(24)" to "(25)."
2-96	In Section 2.11, reference entry 12, there is a slash where a hyphen should be in the publication number.	In Section 2.11, reference entry 12, changed "FHWA/SA-90-017" to "FHWA-SA-90-017."
	In Section 2.11, reference entry 14, there is an equal sign where a hyphen should be in the publication number.	In Section 2.11, reference entry 14, changed "FHWA-SA=14-015" to "FHWA-SA-14-015."
Chapter 3		
3-38	U.S. Customary Equation 3-11 shows a multiplier of "1.15" in the denominator: $R_{\text{PI}} = \frac{V_R^2}{1.15e_{\max}}$	In U.S. Customary Equation 3-11, changed multiplier in the denominator to "0.15": $R_{\text{PI}} = \frac{V_R^2}{0.15e_{\max}}$
3-49	For $e_{\max}$ = 12%, e = 5.8%, and V = 45 mph, Table 3-12 shows a radius of 1,920 ft.	In Table 3-12, for $e_{\max}$ = 12%, e = 5.8%, and V = 45 mph, changed the radius from 1,920 ft to 1,620 ft.
3-66 through 3-69	In Table 3-16 U.S. Customary and metric, table values were not updated to match the gradient change.	In Table 3-16 U.S. Customary and metric, updated table values to match the gradient change.
3-83	In Figure 3-8(c), object B and its centerline(s) are slightly out of position.	In Figure 3-8(c), corrected positioning of object B and its centerline.
3-85	In Section 3.3.8.7, paragraph 1, some values are inconsistent with text elsewhere in the chapter.	Revised to read "Even when the maximum relative gradient is used to define runoff length, the length of vertical curve does not need to be large to conform to the 0.67 percent break at the 30-mph [50-km/h] design speed (see Figure 3-8) and 0.50 percent break at design speeds of 50 mph [80 km/h] and higher. Where the traveled way is revolved about an edge, these grade breaks are doubled to 1.33 percent for the 30-mph [50-km/h] design speed and to 1.00 percent for design speeds of 50 mph [80 km/h] and higher."
	In Section 3.3.8.7, paragraph 1, second to last sentence, metric values should be listed second.	Revised to read "For an approximate guide, however, the minimum vertical curve length in feet [meters] can be used as numerically equal to the design speed in miles per hour [0.2 times the design speed in kilometers per hour]."
3-93	In both the U.S. Customary and metric versions of Equation 3-33, the square root symbol extends too far, including " $-R$" as part of the square root expression.	Corrected both the U.S. Customary and metric versions of Equation 3-33 so that " $- R$" is not part of the square root expression.

Page	Existing	Corrected
3-96	U.S. Customary Equation 3-34 shows "0.1" just after the equal sign: $Z = 0.1\left(V / \sqrt{R}\right)$	In U.S. Customary Equation 3-34, deleted "0.1" just after the equal sign: $Z = \left(V / \sqrt{R}\right)$
3-99	The first row of Table 3-24a has one cell too many highlighted.	In the first row of Table 3-24a, for R = 7000 ft, removed the highlighting from the next to last cell on the right, where the value is 2.0.
3-123	In paragraph 3, second to last sentence, the percentage upgrade is incorrect in "maintain a minimum speed of 50 mph [80 km/h] on a 3 percent upgrade."	In paragraph 3, second to last sentence, changed the percentage upgrade to read "maintain a minimum speed of 50 mph [80 km/h] on a 2 percent upgrade."
3-124	Metric Figure 3-15 is incorrectly positioned.	Rotated metric Figure 3-15 by 90 degrees.
3-125	Metric Figure 3-16 has a y axis label of "mph."	Changed metric Figure 3-16's y axis label to "km/h."
3-126	In paragraph 1, all figure callouts are incorrect.	Changed figure callouts as follows: "3-16" to "3-15," "3-17" to "3-16," "3-18" to "3-17," and "3-19" to "3-18."
3-140	In the last paragraph, the callout is "Figure 3-18."	Changed the callout from "Figure 3-18" to "Figure 3-17."
3-184	In Figure 3-40(C), the label "Preferred" has been separated from its arrow, which has been separated from its point of interest: the dashed line.	In Figure 3-40(C), moved the label "Preferred" and the arrow to indicate the dashed line.
3-185	In Figure 3-40(H), the label "Minimum Curve…" is set on top of the arrow.	In Figure 3-40(H), moved the label "Minimum Curve…" into the clear.
	In Figure 3-40(I), "conves" is a typographical error.	In Figure 3-40(I), changed "conves" to "convex."
	In Figure 3-40(J), "Verticles" is a typographical error.	In Figure 3-40(J), changed "Verticles" to "Vertices."
	In Figure 3-40(L), "shrot" is a typographical error.	In Figure 3-40(L), changed "shrot" to "short."
Chapter 4		
4-89	In Section 4.21, reference entry 18, the publication number and year are incorrect.	In Section 4.21, reference entry 18, changed "FHWA-SA-96-078" to "FHWA-NHI-10-009" and the publication year from 1996 to 2009.
4-92	In Section 4.21, reference entry 37, hyphens are missing from the publication number.	In Section 4.21, reference entry 37, changed "FHWA NHI 01-002" to "FHWA-NHI-01-002."
Chapter 5		
5-40	In Section 5.7, reference entry 7, the title is incomplete.	In Section 5.7, reference entry 7, changed the title from "*Drainage Manual*" to "*AASHTO Drainage Manual*."

Page	Existing	Corrected
Chapter 6		
6-25	In Section 6.4, reference entry 7, the title is incomplete.	In Section 6.4, reference entry 7, changed the title from "*Drainage Manual*" to "*AASHTO Drainage Manual*."
Chapter 7		
7-71	In Section 7.4, reference entry 11, the publication number has a zero where a hyphen should be.	In Section 7.4, reference entry 11, changed "FHWA-HEP017-024" to "FHWA-HEP-17-024."
Chapter 8		
8-4	Section 8.2.4 did not allow narrower shoulders in mountainous terrain as is allowed in the Interstate standard.	Added the following paragraph to Section 8.2.4: When necessary for freeways in mountainous terrain, the paved right shoulder may be reduced to 8 ft [2.4 m]. On four- or six-lane freeways, the paved left shoulder width may be reduced to 4 ft [1.2 m]. On freeways with eight or more lanes in mountainous terrain, the paved left and right shoulders should be at least 8 ft [2.4 m].
Chapter 9		
9-30	Figure 9-13, Typical Single-Lane Roundabout, is out of date.	Replaced Figure 9-13, Typical Single-Lane Roundabout.
9-31	Figure 9-14, Typical Multilane Roundabout, is out of date.	Replaced Figure 9-14, Typical Multilane Roundabout.
9-38	In Section 9.5.2.2, paragraph 1, sentence 3 is incomplete.	In Section 9.5.2.2, paragraph 1, added the following language to the end of sentence 3: "from which stopped vehicles may enter or cross a major road on which traffic is not required to stop."
9-97	In both the U.S. Customary and metric versions of Equation 9-3, the variable "t_c" in the denominator is incorrect.	In both the U.S. Customary and metric versions of Equation 9-3, changed the variable "t_c" in the denominator to "t_f."
9-144	Figure 9-61, Basic Geometric Elements of a Roundabout, is out of date.	Replaced Figure 9-61, Basic Geometric Elements of a Roundabout.
9-148	Figure 9-62, Roundabout Lane Configuration Example, is out of date.	Replaced Figure 9-62, Roundabout Lane Configuration Example.
9-156	In Section 9.11.4, the last reference callout is incorrect.	In Section 9.11.4, changed the last reference callout from "(50)" to "(51)."
Chapter 10		
10-92	In Figure 10-55(B1), the outside edges of lane lines are missing.	In Figure 10-55(B1), replaced the missing outside edges of lane lines.

derpasses may be potential crime areas, lessening their usage. The FHWA publication entitled *Informational Report on Lighting Design for Midblock Crosswalks* (*24*) provides information on nighttime visibility needs for pedestrians crossing roadways at nonintersection locations.

A pedestrian's age is an important factor that may explain behavior that leads to collisions between motor vehicles and pedestrians. Very young pedestrians are often careless in traffic from either inexperience or exuberance, whereas older pedestrians may be affected by limitations in sensory, perceptual, cognitive, or motor skills. Driver behavior, such as turning right on red without coming to a complete stop or parking too close to an intersection, may result in collisions with pedestrians. Pedestrian collisions can also be related to the lack of sidewalks, which may force pedestrians to share the traveled way with motorists. Therefore, sidewalk construction should be considered as part of any street improvement in the suburban, urban, and urban core contexts.

Measures with the potential to reduce vehicle–pedestrian crashes and increase pedestrian comfort in the walking environment:

- Use simple designs that minimize crossing widths and minimize the use of more complex elements such as channelization and separate turning lanes.
- Provide curb extensions (bulb-outs) at intersections.
- Assume lower walking speeds.
- Provide median refuge islands of sufficient width at wide intersections.
- Provide lighting and eliminate glare sources at locations that demand multiple information gathering and processing.
- Consider the traffic control system in the context of the geometric design to provide compatibility and adequate advance warning or guide signs for situations that could surprise older drivers or pedestrians or increase their crash frequencies.
- Use accessible pedestrian signals to provide audible and vibrotactile information.
- Consider increasing sign letter size and retroreflectivity to accommodate individuals with decreased visual acuity.
- Use advance yield/stop signs.
- Provide enhanced markings and delineation.
- Use repetition and redundancy in design and in signing.

For further information on older pedestrians and drivers, refer to the FHWA publications, *Handbook for Designing Roadways for the Aging Population* (*14*) and *Pedestrian Safety Guide and Countermeasure Selection System* (*22*).

2.6.3 Walking Speeds

Air temperature, time of day, trip purpose, age, gender, ability, grade, and presence of ice and snow all affect pedestrian walking speeds. Typical pedestrian walking speeds range from approximately 3.0 to 4.0 ft/s [0.9 to 1.2 m/s] (*25*). Older people will generally walk at speeds in the lower end of this range. To accommodate most pedestrians, a walking speed of 3.5 ft/s [1.1 m/s] is used, with a walking speed of 3.0 ft/s used where older pedestrians are expected.

Intersection design can be directly affected by the assumed walking speed, particularly where pedestrian crossings are controlled by pedestrian signals. The *Manual on Uniform Traffic Control Devices* (MUTCD) (*20*) establishes a two-fold process for calculating pedestrian crossing times and distances. First, the pedestrian clearance time (Flashing Don't Walk) is based on a walking speed of 3.5 ft/s [1.1 m/s] measured from curb to curb. Second, the total pedestrian crossing phase (Walk plus Flashing Don't Walk) is calculated using a walking speed of 3.0 ft/s [0.9 m/s] for a crossing measured from the top of the sidewalk ramp to the far curb. These pedestrian walking speeds used in the MUTCD have implications for geometric design because shortening the crossing distance by using curb bulb-outs or narrower lanes can reduce the time for the pedestrian walk phase, thereby increasing the time available for opposing vehicular travel.

2.6.4 Walkway Level of Service

Walking speeds decrease as the pedestrian density of the walkway increases. As with roadway capacities, there is an optimum speed and density under which the walkway will carry the largest volume. The width used for walkway calculations should be reduced where parking meters, hydrants, newsstands, litter barrels, utility poles, or similar obstructions preclude the use of the full walkway. For a more detailed analysis of sidewalk, stairway, and crosswalk design and capacities, see the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*2*) and the *Highway Capacity Manual* (*43*).

2.6.5 Intersections

When pedestrians encounter an intersection, there is a major interruption in pedestrian flow. The sidewalk should provide sufficient storage area for those waiting to cross as well as an area for pedestrian cross traffic to pass.

Once pedestrians are given the walk indication, the crosswalk width and length become important. Crosswalks should be wide enough to accommodate the pedestrian flow in both directions within the duration of the pedestrian signal phase. The wider the street, the longer it takes a pedestrian to cross and proportionately less green signal time will be available for the primary street movements. Additionally, the longer the pedestrian crossing time, the longer the exposure to potential pedestrian–vehicular conflicts.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

method in the first edition of the HSM does not address every facility type and design feature of potential interest and does not consider potential interactions between design features. Still, the HSM represents an important step toward a performance-based project development process. The FHWA IHSDM (*21*) provides a software tool to implement the HSM Part C procedures.

2.10 ENVIRONMENT

A roadway has wide-ranging effects in addition to providing traffic service to users. It is essential that the highway be considered as an element of the total environment. The term "environment," as used here refers to the totality of humankind's surroundings: social, physical, natural, and synthetic. It includes the human, animal, and plant communities and the forces that act on all three. The roadway can and should be located and designed to complement its environment and serve as a catalyst to environmental improvement.

The area surrounding a proposed road or street is an interrelated system of natural, synthetic, and sociologic variables. Changes in one variable within this system cannot be made without some effect on other variables. The consequences of some of these effects may be negligible, but others may have a strong and lasting impact on the environment, including sustaining and improving the quality of human life. Because roadway location and design decisions affect the development of adjacent areas, it is important that environmental variables be given full consideration. Also, care should be exercised so that applicable local, state, and Federal environmental requirements are met.

2.11 REFERENCES

1. AASHTO. *Highway Design and Operational Practices Related to Highway Safety*. American Association of State Highway and Transportation Officials, Washington, DC, 1974.

2. AASHTO. *Guide for the Planning, Design, and Operation of Pedestrian Facilities*, First Edition, GPF-1. American Association of State Highway and Transportation Officials, Washington, DC, 2004.

3. AASHTO. *Strategic Highway Safety Plan—A Comprehensive Plan to Substantially Reduce Vehicle-Related Fatalities and Injuries on the Nation's Highways*, SHSP-2. American Association of State Highway and Transportation Officials, Washington, DC, 2004.

4. AASHTO. *Manual for Assessing Safety Hardware*, Second Edition, MASH-2. American Association of State Highway and Transportation Officials, Washington, DC, 2016.

5. AASHTO. *Highway Safety Manual*, First Edition, HSM-1. American Association of State Highway and Transportation Officials, Washington, DC, 2010.

6. AASHTO. *Roadside Design Guide*, Fourth Edition, RSDG-4. American Association of State Highway and Transportation Officials, Washington, DC, 2011.

7. AASHTO. *Guide for the Development of Bicycle Facilities*, Fourth Edition, GBF-4. American Association of State Highway and Transportation Officials, Washington, DC, 2012.

8. AASHTO. *Guide for Geometric Design of Transit Facilities on Highways and Streets*, First Edition, TVF-1. American Association of State Highway and Transportation Officials, Washington, DC, 2014.

9. AASHTO. *Highway Safety Manual: Supplement*, HSM-1S. American Association of State Highway and Transportation Officials, Washington, DC, 2014.

10. AASHTO. AASHTOWare *Safety Analyst* software. Available at

    www.aashtoware.org and www.safetyanalyst.org

11. Alexander, G. J., and H. Lunenfeld. *Driver Expectancy in Highway Design and Traffic Operations*. FHWA-TO-86-1. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, May 1986. Available at

    https://rosap.ntl.bts.gov/view/dot/890

12. Alexander, G. H. and H. Lunenfeld. *A User's Guide to Positive Guidance*, Third Edition. FHWA-SA-90-017. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1990.

13. Bonneson, J. A. and P. T. McCoy. *National Cooperative Highway Research Program Report 395: Capacity and Operational Effects of Midblock Left-Turn Lanes*. NCHRP, Transportation Research Board, Washington, DC, 1997. Available at

    http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_395.pdf

14. Brewer, M., D. Murillo, and A. Pate, *Handbook for Designing Roadways for the Aging Population*, FHWA-SA-14-015, FHWA, Washington, DC, June 2014.

15. Campbell, J. L, M. G. Lichty, J. L. Brown, C. M. Richard, J. S. Graving, J. Graham, M. O'Laughlin, D. Torbic, and D. Harwood, *National Cooperative Highway Research Program Report 600: Human Factors Guidelines for Road Systems*, Second Edition, NCHRP, TRB, Washington, DC, 2012.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

35. Neuner, M., J. Atkinson, B. Chandler, S. Hallmark, R. Milstead, and R. Retting. *Integrating Speed Management within Roadway Departure, Intersections, and Pedestrian and Bicyclist Safety Focus Areas*, FHWA-SA-16-017, FHWA, Washington, DC, April 2016.

36. Olson, P. L., D. E. Cleveland, P. S. Fancher, L. P. Kostyniuk, and L. W. Schneider. *National Cooperative Highway Research Program Report 270: Parameters Affecting Stopping Sight Distance*. NCHRP, Transportation Research Board, Washington, DC, 1984.

37. Schmidt, I. and P. D. Connolly. Visual Considerations of Man, the Vehicle and the Highways. *Paper No. SP-279-SAE*. Society of Automotive Engineers, New York, 1966. Available for purchase from SAE International at

    http://papers.sae.org/660164

38. Staplin, L., K. Lococo, S. Byington, and D. Harkey. *Highway Design Handbook for Older Drivers and Pedestrians*. FHWA-RD-01-103. Federal Highway Administration, U.S. Department of Transportation, McLean, VA, May 2001. Available at

    http://www.fhwa.dot.gov/publications/research/safety/humanfac/01103/

39. Tilley, D. H., C. W. Erwin, and D. T. Gianturco. Drowsiness and Driving: Preliminary Report of a Population Survey. *Paper No. 730121-SAE*. Society of Automotive Engineers, New York, 1973. Available for purchase from SAE International at http://papers.sae.org/730121

40. TRB. Human Factors and Safety Research Related to Highway Design and Operations Records. In *Transportation Research Record 1281*. Transportation Research Board, National Research Council, Washington, DC, 1990. Available at

    http://pubsindex.trb.org/view.aspx?id=364894

41. TRB. *National Cooperative Highway Research Program Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan*. Transportation Research Board, National Research Council, Washington, DC. Available at

    http://www.trb.org/Main/Blurbs/NCHRP_Report_500_Guidance_for_Implementation_of_th_152868.aspx

    Volume 1: A Guide for Addressing Aggressive-Driving Collisions, 2003

    Volume 2: A Guide for Addressing Collisions Involving Unlicensed Drivers and Drivers with Suspending or Revoked Licenses, 2003

    Volume 3: A Guide for Addressing Collisions with Trees in Hazardous Locations, 2003

Volume 4: A Guide for Addressing Head-On Collisions, 2003

Volume 5: A Guide for Addressing Unsignalized Intersection Collisions, 2003

Volume 6: A Guide for Addressing Run-off-Road Collisions, 2003

Volume 7: A Guide for Reducing Collisions on Horizontal Curves, 2004

Volume 8: A Guide for Reducing Collisions Involving Utility Poles, 2004

Volume 9: A Guide for Reducing Collisions Involving Older Drivers, 2004

Volume 10: A Guide for Reducing Collisions Involving Pedestrians, 2004

Volume 11: A Guide for Increasing Seatbelt Use, 2004

Volume 12: A Guide for Reducing Collisions at Signalized Intersections, 2004

Volume 13: A Guide for Reducing Collisions Involving Heavy Trucks, 2004

Volume 14: A Guide for Reducing Crashes Involving Drowsy and Distracted Drivers, 2005

Volume 15: A Guide for Enhancing Rural Emergency Medical Services, 2005

Volume 16: A Guide for Reducing Alcohol-Related Collisions, 2005

Volume 17: A Guide for Reducing Work Zone Collisions, 2005

Volume 18: A Guide for Reducing Collisions Involving Bicycles, 2008

Volume 19: A Guide for Reducing Collisions Involving Young Drivers, 2007

Volume 20: A Guide for Reducing Head-On Crashes on Freeways, 2008

Volume 21: Safety Data and Analysis in Developing Emphasis Area Plans, 2008

Volume 22: A Guide for Addressing Collisions Involving Motorcycles, 2008

Volume 23: A Guide for Reducing Speed-Related Crashes, 2009

42. TRB. *Access Management Manual*. Transportation Research Board, National Research Council, Washington, DC, 2015 or most recent edition.

43. TRB. *Highway Capacity Manual: A Guide for Multimodal Mobility Analysis*. Sixth Edition. Transportation Research Board, National Research Council, Washington, DC, 2016.

44. T. Y. Lin International. *Comparison of Turning Radius, Specifications, and Measurements for a 45' Bus*. Golden Gate Bridge Highway & Transportation District and Metropolitan Transportation Commission, San Francisco, CA, 2005.

3.3.4.2 Superelevation

Method 5, described previously, is recommended for the distribution of *e* and *f* for all curves with radii greater than the minimum radius of curvature on highways in rural areas, freeways in urban areas, and high-speed streets in urban areas. Use of Method 5 is discussed in the following text and figures.

3.3.4.3 Procedure for Development of Method 5 Superelevation Distribution

The side friction factors shown as the solid line on Figure 3-4 represent the maximum *f* values selected for design for each speed. When these values are used in conjunction with the recommended Method 5, they determine the *f* distribution curves for the various speeds. Subtracting these computed *f* values from the computed value of *e*/100 + *f* at the design speed, the finalized *e* distribution is thus obtained (see Figure 3-6).

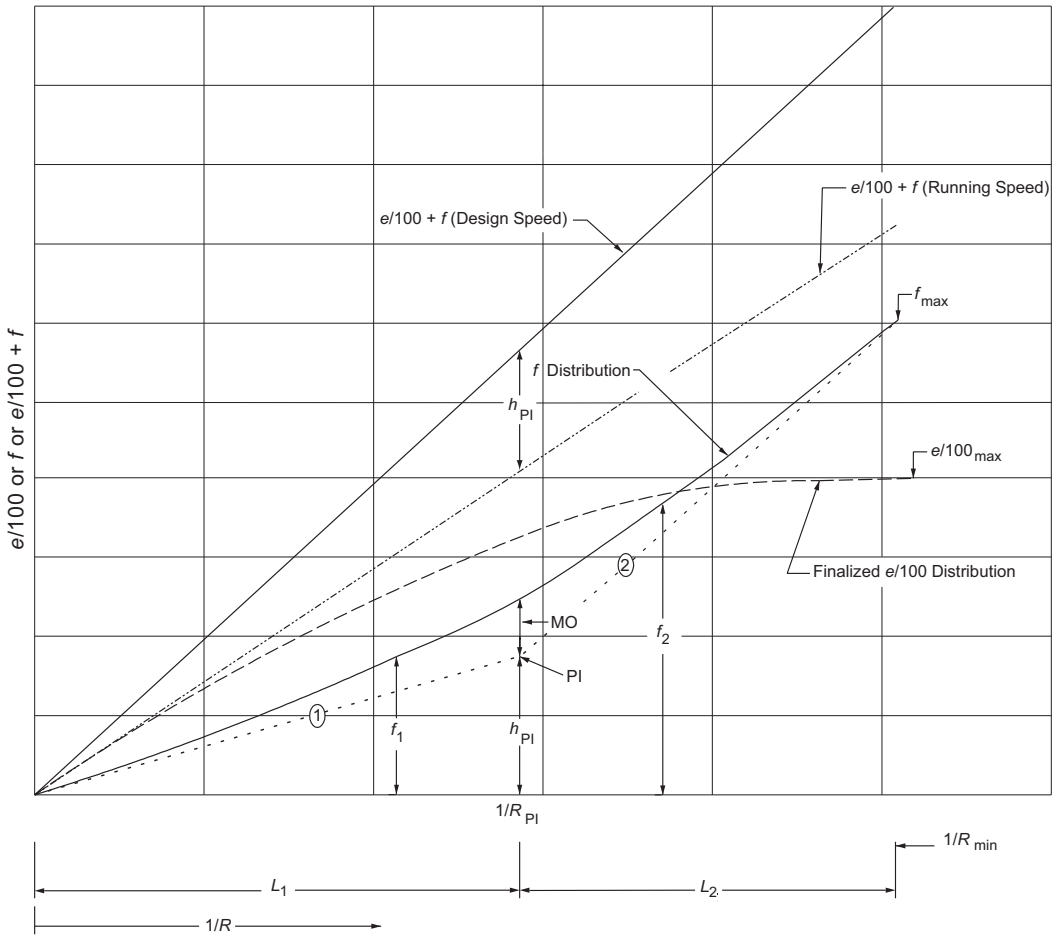


Figure 3-6. Method 5 Procedure for Development of the Superelevation Distribution

The *e* and *f* distributions for Method 5 may be derived using the basic curve equation, neglecting the (1 –0.01*ef*) term as discussed earlier in this chapter, using the following sequence of equations:

U.S. Customary	Metric	
$0.01e + f = \frac{V^2}{15R}$	$0.01e + f = \frac{V^2}{127R}$	(3-9)
where:	where:	
$V = VD$ = design speed, mph	$V = VD$ = design speed, km/h	
$e = e_{max}$ = maximum superelevation, percent	$e = e_{max}$ = maximum superelevation, percent	
$f = f_{max}$ = maximum allowable side friction factor	$f = f_{max}$ = maximum allowable side friction factor	
$R = R_{min}$ = minimum radius, ft	$R = R_{min}$ = minimum radius, m	
then:	then:	
$R_{min} = \frac{V_D^2}{15(0.01e_{max} + f_{max})}$	$R_{min} = \frac{V_D^2}{127(0.01e_{max} + f_{max})}$	(3-10)
and where:	and where:	
$V = V_R$ = running speed, mph	$V = V_R$ = running speed, km/h	
$R = R_{PI}$ = radius at the Point of Intersection, PI, of legs (1) and (2) of the *f* distribution parabolic curve (= *R* at the point of intersection of $0.01e_{max}$ and $(0.01e + f)_R$)	$R = R_{PI}$ = radius at the Point of Intersection, PI, of legs (1) and (2) of the *f* distribution parabolic curve (= *R* at the point of intersection of $0.01e_{max}$ and $(0.01e + f)_R$)	
then:	then:	
$R_{PI} = \frac{V_R^2}{0.15e_{max}}$	$R_{PI} = \frac{V_R^2}{1.27e_{max}}$	(3-11)
Because $(0.01e + f)_D - (0.01e + f)_R = h$, at point R_{PI} the equations reduce to the following:	Because $(0.01e + f)_D - (0.01e + f)_R = h$, at point R_{PI} the equations reduce to the following:	
$h_{PI} = \frac{(0.01e_{max})\ V_D^2}{V_R^2} - 0.01e_{max}$	$h_{PI} = \frac{(0.01e_{max})\ V_D^2}{V_R^2} - 0.01e_{max}$	(3-12)
where h_{PI} = PI offset from the 1/*R* axis.	where h_{PI} = PI offset from the 1/*R* axis.	
Also:	Also:	
$S_1 = \frac{h_{PI}(R_{PI})}{5729.58}$	$S_1 = h_{PI}(R_{PI})$	(3-13)

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Table 3-12. Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 12%

U.S. Customary														
e (%)	V_d = 15 mph	V_d = 20 mph	V_d = 25 mph	V_d = 30 mph	V_d = 35 mph	V_d = 40 mph	V_d = 45 mph	V_d = 50 mph	V_d = 55 mph	V_d = 60 mph	V_d = 65 mph	V_d = 70 mph	V_d = 75 mph	V_d = 80 mph
	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)
NC	950	1690	2460	3370	4390	5580	6910	8370	9990	11800	13200	14800	16400	18100
RC	700	1250	1820	2490	3260	4140	5130	6220	7430	8740	9840	11000	12300	13600
2.2	631	1130	1640	2250	2950	3750	4640	5640	6730	7930	8920	9980	11200	12400
2.4	574	1030	1500	2060	2690	3420	4240	5150	6150	7240	8160	9130	10200	11300
2.6	526	936	1370	1890	2470	3140	3900	4730	5660	6670	7510	8420	9380	10500
2.8	484	863	1270	1740	2280	2910	3600	4380	5240	6170	6960	7800	8700	9660
3.0	448	799	1170	1620	2120	2700	3350	4070	4870	5740	6480	7270	8110	9010
3.2	417	743	1090	1510	1970	2520	3130	3800	4550	5370	6060	6800	7600	8440
3.4	389	693	1020	1410	1850	2360	2930	3560	4270	5030	5690	6390	7140	7940
3.6	364	649	953	1320	1730	2220	2750	3350	4020	4740	5360	6020	6740	7500
3.8	341	610	896	1250	1630	2090	2600	3160	3790	4470	5060	5700	6380	7100
4.0	321	574	845	1180	1540	1980	2460	2990	3590	4240	4800	5400	6050	6740
4.2	303	542	798	1110	1460	1870	2330	2840	3400	4020	4560	5130	5750	6420
4.4	286	512	756	1050	1390	1780	2210	2700	3240	3830	4340	4890	5490	6120
4.6	271	485	717	997	1320	1690	2110	2570	3080	3650	4140	4670	5240	5850
4.8	257	460	681	948	1260	1610	2010	2450	2940	3480	3960	4470	5020	5610
5.0	243	437	648	904	1200	1540	1920	2340	2810	3330	3790	4280	4810	5380
5.2	231	415	618	862	1140	1470	1840	2240	2700	3190	3630	4110	4620	5170
5.4	220	395	589	824	1090	1410	1760	2150	2590	3060	3490	3950	4440	4980
5.6	209	377	563	788	1050	1350	1690	2060	2480	2940	3360	3800	4280	4800
5.8	199	359	538	754	1000	1300	1620	1980	2390	2830	3230	3660	4130	4630
6.0	190	343	514	723	960	1250	1560	1910	2300	2730	3110	3530	3990	4470
6.2	181	327	492	694	922	1200	1500	1840	2210	2630	3010	3410	3850	4330
6.4	172	312	471	666	886	1150	1440	1770	2140	2540	2900	3300	3730	4190
6.6	164	298	452	639	852	1110	1390	1710	2060	2450	2810	3190	3610	4060
6.8	156	284	433	615	820	1070	1340	1650	1990	2370	2720	3090	3500	3940
7.0	148	271	415	591	790	1030	1300	1590	1930	2290	2630	3000	3400	3820
7.2	140	258	398	568	762	994	1250	1540	1860	2220	2550	2910	3300	3720
7.4	133	246	382	547	734	960	1210	1490	1810	2150	2470	2820	3200	3610
7.6	125	234	366	527	708	928	1170	1440	1750	2090	2400	2740	3120	3520
7.8	118	222	351	507	684	897	1130	1400	1700	2020	2330	2670	3030	3430
8.0	111	210	336	488	660	868	1100	1360	1650	1970	2270	2600	2950	3340
8.2	105	199	321	470	637	840	1070	1320	1600	1910	2210	2530	2880	3260
8.4	100	190	307	452	615	813	1030	1280	1550	1860	2150	2460	2800	3180
8.6	95	180	294	435	594	787	997	1240	1510	1810	2090	2400	2740	3100
8.8	90	172	281	418	574	762	967	1200	1470	1760	2040	2340	2670	3030
9.0	85	164	270	403	554	738	938	1170	1430	1710	1980	2280	2610	2960
9.2	81	156	259	388	535	715	910	1140	1390	1660	1940	2230	2550	2890
9.4	77	149	248	373	516	693	883	1100	1350	1620	1890	2180	2490	2830

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Table 3-12. Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 12% (Continued)

U.S. Customary														
e (%)	V_d = 15 mph	V_d = 20 mph	V_d = 25 mph	V_d = 30 mph	V_d = 35 mph	V_d = 40 mph	V_d = 45 mph	V_d = 50 mph	V_d = 55 mph	V_d = 60 mph	V_d = 65 mph	V_d = 70 mph	V_d = 75 mph	V_d = 80 mph
	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)	R (ft)
9.6	74	142	238	359	499	671	857	1070	1310	1580	1840	2130	2440	2770
9.8	70	136	228	346	481	650	832	1040	1280	1540	1800	2080	2380	2710
10.0	67	130	219	333	465	629	806	1010	1250	1500	1760	2030	2330	2660
10.2	64	124	210	320	448	608	781	980	1210	1460	1720	1990	2280	2600
10.4	61	118	201	308	432	588	757	951	1180	1430	1680	1940	2240	2550
10.6	58	113	192	296	416	568	732	922	1140	1390	1640	1900	2190	2500
10.8	55	108	184	284	400	548	707	892	1110	1350	1600	1860	2150	2460
11.0	52	102	175	272	384	527	682	862	1070	1310	1560	1820	2110	2410
11.2	49	97	167	259	368	506	656	831	1040	1270	1510	1780	2070	2370
11.4	47	92	158	247	351	485	629	799	995	1220	1470	1730	2020	2320
11.6	44	86	149	233	333	461	600	763	953	1170	1410	1680	1970	2280
11.8	40	80	139	218	312	434	566	722	904	1120	1350	1620	1910	2230
12.0	34	68	119	188	272	381	500	641	807	1000	1220	1480	1790	2130

Metric												
e (%)	V_d = 20 km/h	V_d = 30 km/h	V_d = 40 km/h	V_d = 50 km/h	V_d = 60 km/h	V_d = 70 km/h	V_d = 80 km/h	V_d = 90 km/h	V_d = 100 km/h	V_d = 110 km/h	V_d = 120 km/h	V_d = 130 km/h
	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)
NC	210	459	804	1130	1540	2030	2510	3040	3720	4280	4990	5440
RC	155	338	594	835	1150	1510	1870	2270	2770	3190	3740	4080
2.2	139	306	536	755	1040	1360	1690	2050	2510	2900	3390	3710
2.4	127	278	488	688	942	1250	1550	1880	2300	2650	3110	3400
2.6	116	255	448	631	865	1140	1420	1730	2110	2440	2860	3140
2.8	107	235	413	583	799	1060	1320	1600	1960	2260	2660	2910
3.0	99	218	382	541	742	980	1220	1490	1820	2110	2480	2720
3.2	92	202	356	504	692	914	1140	1390	1700	1970	2320	2550
3.4	86	189	332	472	648	856	1070	1300	1600	1850	2180	2400
3.6	81	177	312	443	609	805	1010	1230	1510	1750	2060	2270
3.8	76	166	293	417	573	759	947	1160	1420	1650	1950	2150
4.0	71	157	276	393	542	718	896	1100	1350	1560	1850	2040
4.2	67	148	261	372	513	680	850	1040	1280	1490	1760	1940
4.4	64	140	247	353	487	646	808	988	1220	1420	1680	1850
4.6	60	132	234	335	436	615	770	941	1160	1350	1600	1770
4.8	57	126	222	319	441	586	734	899	1110	1290	1530	1700
5.0	54	119	211	304	421	560	702	860	1060	1240	1470	1630

The superelevation runoff lengths given in Table 3-16 are based on 12-ft [3.6-m] lanes. For other lane widths, the appropriate runoff length should vary in proportion to the ratio of the actual lane width to 12 ft [3.6 m]. Shorter lengths could be applied for designs with 10- and 11-ft [3.0- and 3.3-m] lanes, but considerations of consistency and practicality suggest that the runoff lengths for 12-ft [3.6-m] lanes should be used in all cases.

Table 3-16a. Superelevation Runoff L_r (ft) for Horizontal Curves

U.S. Customary																														
e (%)	V_d = 15 mph		V_d = 20 mph		V_d = 25 mph		V_d = 30 mph		V_d = 35 mph		V_d = 40 mph		V_d = 45 mph		V_d = 50 mph		V_d = 55 mph		V_d = 60 mph		V_d = 65 mph		V_d = 70 mph		V_d = 75 mph		V_d = 80 mph		V_d = 85 mph	
	Number of Lanes Rotated. Note that 1 lane rotated is typical for a 2-lane highway; 2 lanes rotated is typical for a 4-lane highway, etc. (See Table 3-15.)																													
	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)
1.5	23	34	23	34	25	37	27	40	29	44	32	47	34	51	36	54	36	54	36	54	36	54	36	54	36	54	36	54	36	54
2.0	30	45	30	45	33	50	36	54	39	59	42	63	45	68	48	72	48	72	48	72	48	72	48	72	48	72	48	72	48	72
2.2	33	50	33	50	36	54	40	59	43	64	46	69	50	74	53	79	53	79	53	79	53	79	53	79	53	79	53	79	53	79
2.4	36	54	36	54	40	59	43	65	47	70	50	76	54	81	58	86	58	86	58	86	58	86	58	86	58	86	58	86	58	86
2.6	39	59	39	59	43	64	47	70	51	76	55	82	59	88	62	94	62	94	62	94	62	94	62	94	62	94	62	94	62	94
2.8	42	63	42	63	46	69	50	76	55	82	59	88	63	95	67	101	67	101	67	101	67	101	67	101	67	101	67	101	67	101
3.0	45	68	45	68	50	74	54	81	59	88	63	95	68	101	72	108	72	108	72	108	72	108	72	108	72	108	72	108	72	108
3.2	48	72	48	72	53	79	58	86	62	94	67	101	72	108	77	115	77	115	77	115	77	115	77	115	77	115	77	115	77	115
3.4	51	77	51	77	56	84	61	92	66	100	71	107	77	115	82	122	82	122	82	122	82	122	82	122	82	122	82	122	82	122
3.6	54	81	54	81	59	89	65	97	70	105	76	113	81	122	86	130	86	130	86	130	86	130	86	130	86	130	86	130	86	130
3.8	57	86	57	86	63	94	68	103	74	111	80	120	86	128	91	137	91	137	91	137	91	137	91	137	91	137	91	137	91	137
4.0	60	90	60	90	66	99	72	108	78	117	84	126	90	135	96	144	96	144	96	144	96	144	96	144	96	144	96	144	96	144
4.2	63	95	63	95	69	104	76	113	82	123	88	132	95	142	101	151	101	151	101	151	101	151	101	151	101	151	101	151	101	151
4.4	66	99	66	99	73	109	79	119	86	129	92	139	99	149	106	158	106	158	106	158	106	158	106	158	106	158	106	158	106	158
4.6	69	104	69	104	76	114	83	124	90	135	97	145	104	155	110	166	110	166	110	166	110	166	110	166	110	166	110	166	110	166
4.8	72	108	72	108	79	119	86	130	94	140	101	151	108	162	115	173	115	173	115	173	115	173	115	173	115	173	115	173	115	173
5.0	75	113	75	113	83	124	90	135	98	146	105	158	113	169	120	180	120	180	120	180	120	180	120	180	120	180	120	180	120	180
5.2	78	117	78	117	86	129	94	140	101	152	109	164	117	176	125	187	125	187	125	187	125	187	125	187	125	187	125	187	125	187
5.4	81	122	81	122	89	134	97	146	105	158	113	170	122	182	130	194	130	194	130	194	130	194	130	194	130	194	130	194	130	194
5.6	84	126	84	126	92	139	101	151	109	164	118	177	126	189	134	202	134	202	134	202	134	202	134	202	134	202	134	202	134	202
5.8	87	131	87	131	96	144	104	157	113	170	122	183	131	196	139	209	139	209	139	209	139	209	139	209	139	209	139	209	139	209
6.0	90	135	90	135	99	149	108	162	117	176	126	189	135	203	144	216	144	216	144	216	144	216	144	216	144	216	144	216	144	216
6.2	93	140	93	140	102	154	112	167	121	181	130	195	140	209	149	223	149	223	149	223	149	223	149	223	149	223	149	223	149	223
6.4	96	144	96	144	106	158	115	173	125	187	135	202	144	216	154	230	154	230	154	230	154	230	154	230	154	230	154	230	154	230
6.6	99	149	99	149	109	163	119	178	129	193	139	208	149	223	158	238	158	238	158	238	158	238	158	238	158	238	158	238	158	238
6.8	102	153	102	153	112	168	122	184	133	199	143	214	153	230	163	245	163	245	163	245	163	245	163	245	163	245	163	245	163	245

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Table 3-16a. Superelevation Runoff L_r (ft) for Horizontal Curves (Continued)

U.S. Customary

e (%)	V_d = 15 mph		V_d = 20 mph		V_d = 25 mph		V_d = 30 mph		V_d = 35 mph		V_d = 40 mph		V_d = 45 mph		V_d = 50 mph		V_d = 55 mph		V_d = 60 mph		V_d = 65 mph		V_d = 70 mph		V_d = 75 mph		V_d = 80 mph		V_d = 85 mph	
	Number of Lanes Rotated. Note that 1 lane rotated is typical for a 2-lane highway; 2 lanes rotated is typical for a 4-lane highway, etc. (See Table 3-15.)																											
	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)
7.0	105	158	105	158	116	173	126	189	137	205	147	221	158	236	168	252	168	252	168	252	168	252	168	252	168	252	168	252
7.2	108	162	108	162	119	178	130	194	140	211	151	227	162	243	173	259	173	259	173	259	173	259	173	259	173	259	173	259
7.4	111	167	111	167	122	183	133	200	144	217	156	233	167	250	178	266	178	266	178	266	178	266	178	266	178	266	178	266
7.6	114	171	114	171	125	188	137	205	148	222	160	240	171	257	182	274	182	274	182	274	182	274	182	274	182	274	182	274
7.8	117	176	117	176	129	193	140	210	152	228	164	246	176	263	187	281	187	281	187	281	187	281	187	281	187	281	187	281
8.0	120	180	120	180	132	198	144	216	156	234	168	252	180	270	192	288	192	288	192	288	192	288	192	288	192	288	192	288
8.2	123	185	123	185	135	203	148	221	160	240	172	258	185	277	197	295	197	295	197	295	197	295	197	295	197	295	197	295
8.4	126	189	126	189	139	208	151	227	164	246	177	265	189	284	202	302	202	302	202	302	202	302	202	302	202	302	202	302
8.6	129	194	129	194	142	213	155	232	168	252	181	271	194	290	206	310	206	310	206	310	206	310	206	310	206	310	206	310
8.8	132	198	132	198	145	218	158	237	172	258	185	277	198	297	211	317	211	317	211	317	211	317	211	317	211	317	211	317
9.0	135	203	135	203	149	223	162	243	176	263	189	284	203	304	216	324	216	324	216	324	216	324	216	324	216	324	216	324
9.2	138	207	138	207	152	228	166	248	180	269	193	290	207	311	221	331	221	331	221	331	221	331	221	331	221	331	221	331
9.4	141	212	141	212	155	233	169	254	183	275	198	296	212	317	226	338	226	338	226	338	226	338	226	338	226	338	226	338
9.6	144	216	144	216	158	238	173	259	187	281	202	303	216	324	230	346	230	346	230	346	230	346	230	346	230	346	230	346
9.8	147	221	147	221	162	243	176	264	191	287	206	309	221	331	235	353	235	353	235	353	235	353	235	353	235	353	235	353
10.0	150	225	150	225	165	248	180	270	195	293	210	315	225	338	240	360	240	360	240	360	240	360	240	360	240	360	240	360
10.2	153	230	153	230	168	253	184	275	199	299	214	322	230	344	245	367	245	367	245	367	245	367	245	367	245	367	245	367
10.4	156	234	156	234	172	257	187	281	203	304	219	328	234	351	250	374	250	374	250	374	250	374	250	374	250	374	250	374
10.6	159	239	159	239	175	262	191	286	207	310	223	334	239	358	254	382	254	382	254	382	254	382	254	382	254	382	254	382
10.8	162	243	162	243	178	267	194	291	211	316	227	340	243	365	259	389	259	389	259	389	259	389	259	389	259	389	259	389
11.0	165	248	165	248	182	272	198	297	215	322	231	347	248	371	264	396	264	396	264	396	264	396	264	396	264	396	264	396
11.2	168	252	168	252	185	277	201	302	219	328	235	353	252	378	269	403	269	403	269	403	269	403	269	403	269	403	269	403
11.4	171	257	171	257	188	282	205	308	222	334	240	359	257	385	274	410	274	410	274	410	274	410	274	410	274	410	274	410
11.6	174	261	174	261	191	287	209	313	226	340	244	366	261	392	278	418	278	418	278	418	278	418	278	418	278	418	278	418
11.8	177	266	177	266	195	292	212	318	230	345	248	372	266	398	283	425	283	425	283	425	283	425	283	425	283	425	283	425
12.0	180	270	180	270	198	297	216	324	234	351	252	378	270	405	288	432	288	432	288	432	288	432	288	432	288	432	288	432

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Table 3-16b. Superelevation Runoff L_r (m) for Horizontal Curves

	Metric																													
e (%)	V_d = 20 km/h		V_d = 30 km/h		V_d = 40 km/h		V_d = 50 km/h		V_d = 60 km/h		V_d = 70 km/h		V_d = 80 km/h		V_d = 90 km/h		V_d = 100 km/h		V_d = 110 km/h		V_d = 120 km/h		V_d = 130 km/h		V_d = 140 km/h	
	Number of Lanes Rotated. Note that 1 lane rotated is typical for a 2-lane highway, 2 lanes rotated is typical for a 4-lane highway, etc. (See Table 3-15.)																											
	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)	L_r(m)
1.5	7	10	7	10	7	11	8	12	9	13	9	14	11	16	11	16	11	16	11	16	11	16	11	16	11	16	11	16	11	16
2.0	9	14	9	14	10	15	11	16	12	18	13	19	14	22	14	22	14	22	14	22	14	22	14	22	14	22	14	22	14	22
2.2	10	15	10	15	11	16	12	18	13	19	14	21	16	24	16	24	16	24	16	24	16	24	16	24	16	24	16	24	16	24
2.4	11	16	11	16	12	18	13	19	14	21	15	23	17	26	17	26	17	26	17	26	17	26	17	26	17	26	17	26	17	26
2.6	12	18	12	18	13	19	14	21	15	23	16	25	19	28	19	28	19	28	19	28	19	28	19	28	19	28	19	28	19	28
2.8	13	19	13	19	14	21	15	23	16	25	18	26	20	30	20	30	20	30	20	30	20	30	20	30	20	30	20	30	20	30
3.0	14	20	14	20	15	22	16	24	18	26	19	28	22	32	22	32	22	32	22	32	22	32	22	32	22	32	22	32	22	32
3.2	14	22	14	22	16	24	17	26	19	28	20	30	23	35	23	35	23	35	23	35	23	35	23	35	23	35	23	35	23	35
3.4	15	23	15	23	17	25	18	28	20	30	21	32	24	37	24	37	24	37	24	37	24	37	24	37	24	37	24	37	24	37
3.6	16	24	16	24	18	27	19	29	21	32	23	34	26	39	26	39	26	39	26	39	26	39	26	39	26	39	26	39	26	39
3.8	17	26	17	26	19	28	21	31	22	33	24	36	27	41	27	41	27	41	27	41	27	41	27	41	27	41	27	41	27	41
4.0	18	27	18	27	20	30	22	32	23	35	25	38	29	43	29	43	29	43	29	43	29	43	29	43	29	43	29	43	29	43
4.2	19	28	19	28	21	31	23	34	25	37	26	40	30	45	30	45	30	45	30	45	30	45	30	45	30	45	30	45	30	45
4.4	20	30	20	30	22	33	24	36	26	39	28	42	32	48	32	48	32	48	32	48	32	48	32	48	32	48	32	48	32	48
4.6	21	31	21	31	23	34	25	37	27	40	29	43	33	50	33	50	33	50	33	50	33	50	33	50	33	50	33	50	33	50
4.8	22	32	22	32	24	36	26	39	28	42	30	45	35	52	35	52	35	52	35	52	35	52	35	52	35	52	35	52	35	52
5.0	23	34	23	34	25	37	27	40	29	44	32	47	36	54	36	54	36	54	36	54	36	54	36	54	36	54	36	54	36	54
5.2	23	35	23	35	26	39	28	42	30	46	33	49	37	56	37	56	37	56	37	56	37	56	37	56	37	56	37	56	37	56
5.4	24	36	24	36	27	40	29	44	32	47	34	51	39	58	39	58	39	58	39	58	39	58	39	58	39	58	39	58	39	58
5.6	25	38	25	38	28	42	30	45	33	49	35	53	40	60	40	60	40	60	40	60	40	60	40	60	40	60	40	60	40	60
5.8	26	39	26	39	29	43	31	47	34	51	37	55	42	63	42	63	42	63	42	63	42	63	42	63	42	63	42	63	42	63
6.0	27	41	27	41	30	45	32	49	35	53	38	57	43	65	43	65	43	65	43	65	43	65	43	65	43	65	43	65	43	65
6.2	28	42	28	42	31	46	33	50	36	54	39	59	45	67	45	67	45	67	45	67	45	67	45	67	45	67	45	67	45	67
6.4	29	43	29	43	32	48	35	52	37	56	40	60	46	69	46	69	46	69	46	69	46	69	46	69	46	69	46	69	46	69
6.6	30	45	30	45	33	49	36	53	39	58	42	62	48	71	48	71	48	71	48	71	48	71	48	71	48	71	48	71	48	71
6.8	31	46	31	46	34	51	37	55	40	60	43	64	49	73	49	73	49	73	49	73	49	73	49	73	49	73	49	73	49	73

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Table 3-16b. Superelevation Runoff L_r (m) for Horizontal Curves (Continued)

Metric																															
e (%)	V_d = 20 km/h		V_d = 30 km/h			V_d = 40 km/h			V_d = 50 km/h			V_d = 60 km/h			V_d = 70 km/h			V_d = 80 km/h			V_d = 90 km/h			V_d = 100 km/h			V_d = 110 km/h			V_d = 120 km/h			V_d = 130 km/h			V_d = 140 km/h	
	Number of Lanes Rotated. Note that 1 lane rotated is typical for a 2-lane highway, 2 lanes rotated is typical for a 4-lane highway, etc. (See Table 3-15.)																														
	1	2	1	2		1	2		1	2		1	2		1	2		1	2		1	2		1	2		1	2		1	2	
	L_r (m)	L_r (m)	L_r (m)	L_r (m)		L_r (m)	L_r (m)		L_r (m)	L_r (m)		L_r (m)	L_r (m)		L_r (m)	L_r (m)		L_r (m)	L_r (m)		L_r (m)	L_r (m)		L_r (m)	L_r (m)		L_r (m)	L_r (m)		L_r (m)	L_r (m)		L_r (m)	L_r (m)	
7.0	32	47	32	47		35	52		38	57		41	61		44	66		50	76		50	76		50	76		50	76		50	76	
7.2	32	49	32	49		36	53		39	58		42	63		45	68		52	78		52	78		52	78		52	78		52	78	
7.4	33	50	33	50		37	55		40	60		43	65		47	70		53	80		53	80		53	80		53	80		53	80	
7.6	34	51	34	51		38	56		41	62		44	67		48	72		55	82		55	82		55	82		55	82		55	82	
7.8	35	53	35	53		39	58		42	63		46	68		49	74		56	84		56	84		56	84		56	84		56	84	
8.0	36	54	36	54		40	59		43	65		47	70		50	76		58	86		58	86		58	86		58	86		58	86	
8.2	37	55	37	55		41	61		44	66		48	72		52	77		59	89		59	89		59	89		59	89		59	89	
8.4	38	57	38	57		42	62		45	68		49	74		53	79		60	91		60	91		60	91		60	91		60	91	
8.6	39	58	39	58		43	64		46	70		50	76		54	81		62	93		62	93		62	93		62	93		62	93	
8.8	40	59	40	59		44	65		47	71		52	77		55	83		63	95		63	95		63	95		63	95		63	95	
9.0	41	61	41	61		45	67		49	73		53	79		57	85		65	97		65	97		65	97		65	97		65	97	
9.2	41	62	41	62		46	68		50	74		54	81		58	87		66	99		66	99		66	99		66	99		66	99	
9.4	42	63	42	63		47	70		51	76		55	83		59	89		68	102		68	102		68	102		68	102		68	102	
9.6	43	65	43	65		48	71		52	78		56	84		60	91		69	104		69	104		69	104		69	104		69	104	
9.8	44	66	44	66		49	73		53	79		57	86		62	93		71	106		71	106		71	106		71	106		71	106	
10.0	45	68	45	68		50	74		54	81		59	88		63	95		72	108		72	108		72	108		72	108		72	108	
10.2	46	69	46	69		51	76		55	83		60	90		64	96		73	110		73	110		73	110		73	110		73	110	
10.4	47	70	47	70		51	77		56	84		61	91		66	98		75	112		75	112		75	112		75	112		75	112	
10.6	48	72	48	72		52	79		57	86		62	93		67	100		76	114		76	114		76	114		76	114		76	114	
10.8	49	73	49	73		53	80		58	87		63	95		68	102		78	117		78	117		78	117		78	117		78	117	
11.0	50	74	50	74		54	82		59	89		64	97		69	104		79	119		79	119		79	119		79	119		79	119	
11.2	50	76	50	76		55	83		60	91		66	98		71	106		81	121		81	121		81	121		81	121		81	121	
11.4	51	77	51	77		56	85		62	92		67	100		72	108		82	123		82	123		82	123		82	123		82	123	
11.6	52	78	52	78		57	86		63	94		68	102		73	110		84	125		84	125		84	125		84	125		84	125	
11.8	53	80	53	80		58	88		64	96		69	104		74	112		85	127		85	127		85	127		85	127		85	127	
12.0	54	81	54	81		59	89		65	97		70	105		76	113		86	130		86	130		86	130		86	130		86	130	

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

3.3.8.2.2 Minimum Length of Tangent Runout

The length of tangent runout is determined by the amount of adverse cross slope to be removed and the rate at which it is removed. To achieve a smooth edge of pavement profile, the rate of removal should equal the relative gradient used to define the superelevation runoff length. Based on this rationale, the following equation should be used to compute the minimum tangent runout length:

U.S. Customary	Metric
$L_t = \frac{e_{NC}}{e_d} L_r$	$L_t = \frac{e_{NC}}{e_d} L_r$
where:	where:
Lt = minimum length of tangent runout, ft	Lt = minimum length of tangent runout, m
eNC = normal cross slope rate, percent	eNC = normal cross slope rate, percent
ed = design superelevation rate, percent	ed = design superelevation rate, percent
Lr = minimum length of superelevation runoff, ft	Lr = minimum length of superelevation runoff, m

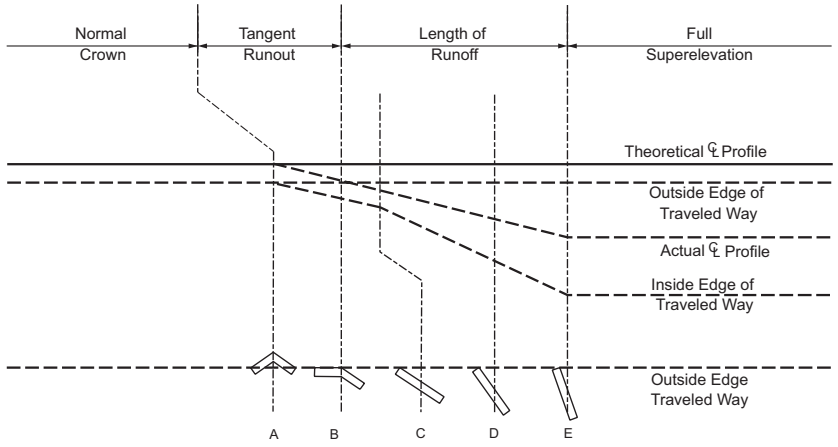
(3-24)

The tangent runout lengths determined with Equation 3-24 are listed in Table 3-16 in the 2.0 percent row.

3.3.8.2.3 Location with Respect to End of Curve

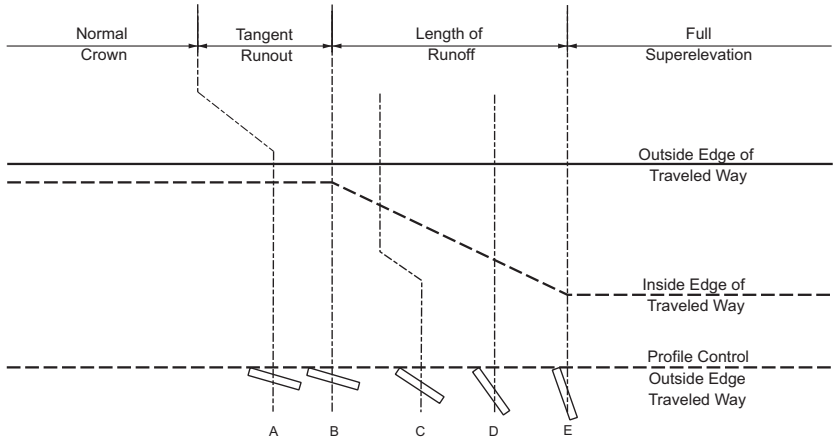
In the tangent-to-curve design, the location of the superelevation runoff length with respect to the Point of Curvature (PC) needs to be determined. Normal practice is to divide the runoff length between the tangent and curved sections and to avoid placing the entire runoff length on either the tangent or the curve. With full superelevation attained at the PC, the runoff lies entirely on the approach tangent, where theoretically no superelevation is needed. At the other extreme, placement of the runoff entirely on the circular curve results in the initial portion of the curve having less than the desired amount of superelevation. Both of these extremes tend to be associated with a large peak lateral acceleration.

Experience indicates that locating a portion of the runoff on the tangent, in advance of the PC, is preferable, in order to limit the peak lateral acceleration and the resulting side friction demand. The magnitude of side friction demand incurred during travel through the runoff can vary with the actual vehicle travel path. Observations indicate that a spiral path results from a driver's natural steering behavior during curve entry or exit. This natural spiral can be assumed to be distributed equally around the PC; as a result, the lateral acceleration incurred at the PC should theoretically be equal to 50 percent of the lateral acceleration associated with the circular curve. Most evidence indicates that the length of this natural spiral ranges from 2- to 4-s travel time; however, its length may also be affected by lane width and the presence of other vehicles.



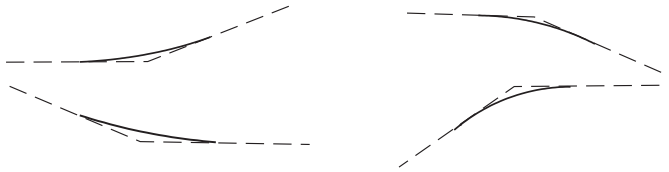
Crowned
Traveled Way Revolved about Outside Edge

– C –



Straight Cross Slope
Traveled Way Revolved about Outside Edge

– D –



Note: Angular breaks to be appropriately rounded as shown (see text)

Figure 3-8. Diagrammatic Profiles Showing Methods of Attaining Superelevation for a Curve to the Right (Continued)

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

The second method, as shown in Figure 3-8B, revolves the traveled way about the inside-edge profile. In this case, the inside-edge profile is determined as a line parallel to the profile reference line. One-half of the change in elevation is made by raising the actual centerline profile with respect to the inside-edge profile and the other half by raising the outside-edge profile an equal amount with respect to the actual centerline profile.

The third method, as shown in Figure 3-8C, revolves the traveled way about the outside-edge profile. This method is similar to that shown in Figure 3-8B except that the elevation change is accomplished below the outside-edge profile instead of above the inside-edge profile.

The fourth method, as shown in Figure 3-8D, revolves the traveled way (having a straight cross slope) about the outside-edge profile. This method is often used for two-lane one-way roadways where the axis of rotation coincides with the edge of the traveled way adjacent to the highway median.

The methods for attaining superelevation are nearly the same for all four methods. Cross section A at one end of the tangent runout is a normal (or straight) cross slope section. At cross section B, the other end of the tangent runout and the beginning of the superelevation runoff, the lane or lanes on the outside of the curve are made horizontal (or level) with the actual centerline profile for Figures 3-8A, 3-8B, and 3-8C; there is no change in cross slope for Figure 3-8D.

At cross section C, the traveled way is a plane, superelevated at the normal cross slope rate. Between cross sections B and C for Figures 3-8A, 3-8B, and 3-8C, the outside lane or lanes change from a level condition to one of superelevation at the normal cross slope rate and normal cross slope is retained on the inner lanes. There is no change between cross sections B and C for Figure 3-8D. Between cross sections C and E the pavement section is revolved to the full rate of superelevation. The rate of cross slope at an intermediate point (e.g., cross section D) is proportional to the distance from cross section C.

In an overall sense, the method of rotation about the centerline shown in Figure 3-8A is usually the most adaptable. On the other hand, the method shown in Figure 3-8B is preferable where the lower edge profile is a major control, as for drainage. With uniform profile conditions, its use results in the greatest distortion of the upper edge profile. Where the overall appearance is a high priority, the methods of Figures 3-8C and 3-8D are desirable because the upper edge profile—the edge most noticeable to drivers—retains the smoothness of the control profile. Thus, the shape and direction of the centerline profile may determine the preferred method for attaining superelevation.

Considering the vast number of profile arrangements that are possible and in recognition of specific issues such as drainage, avoidance of critical grades, aesthetics, and fitting the roadway to the adjacent topography, no general recommendation can be made for adopting any particular axis of rotation. To obtain the most pleasing and functional results, each superelevation transi-

tion section should be considered individually. In practice, any of the pavement reference lines used for the axis of rotation may be best suited for the situation at hand.

3.3.8.7 Design of Smooth Profiles for Traveled-Way Edges

In the diagrammatic profiles shown in Figure 3-8, the tangent profile control lines result in angular breaks at cross sections A, C, and E. For general appearance and safety, these breaks should be rounded in final design by insertion of vertical curves. Angular breaks will be particularly noticeable where hard surfaces, such as concrete barrier or retaining wall, follow the edge of pavement profile. Even when the maximum relative gradient is used to define runoff length, the length of vertical curve does not need to be large to conform to the 0.67 percent break at the 30-mph [50-km/h] design speed (see Figure 3-8) and the 0.50 percent break at design speeds of 50 mph [80 km/h] and higher. Where the traveled way is revolved about an edge, these grade breaks are doubled to 1.33 percent for the 30-mph [50-km/h] design speed and to 1.00 percent for design speeds of 50 mph [80 km/h] and higher. Greater lengths of vertical curve are obviously needed in these cases. Specific criteria have not been established for the lengths of vertical curves at the breaks in the diagrammatic profiles. For an approximate guide, however, the minimum vertical curve length in feet [meters] can be used as numerically equal to the design speed in miles per hour [0.2 times the design speed in kilometers per hour]. Greater lengths should be used where practical as the general profile condition may determine.

A second method uses a graphical approach to define the edge profile. The method essentially is one of spline-line development. In this method, the centerline or other base profile, which usually is computed, is plotted on an appropriate vertical scale. Superelevation control points are in the form of the break points shown in Figure 3-8. Then by means of a spline, curve template, ship curve, or circular curve, smooth-flowing lines are drawn to approximate the straight-line controls. The natural bending of the spline nearly always satisfies the need for minimum smoothing. Once the edge profiles are drawn in the proper relation to one another, elevations can be read at the appropriate intervals (as needed for construction control).

An important advantage of the graphical or spline-line method is the study alternatives it affords the designer. Alternate profile solutions can be developed expeditiously. The net result is a design that is well suited to the particular control conditions. The engineering design labor needed for this procedure is minimal. These several advantages make this method preferable to the other methods of developing profile details for runoff sections.

Divided highways warrant a greater refinement in design and greater attention to appearance than do two-lane highways because divided highways usually serve much greater traffic volumes. Moreover, the cost of such refinements is insignificant compared with the construction cost of the divided highway. Accordingly, there should be greater emphasis on the development of smooth-flowing traveled-way edge profiles for divided highways.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

3.3.8.8 Axis of Rotation with a Median

In the design of divided highways, streets, and parkways, the inclusion of a median in the cross section influences the superelevation transition design. This influence stems from the several possible locations for the axis of rotation. The most appropriate location for this axis depends on the width of the median and its cross section. Common combinations of these factors and the appropriate corresponding axis location are described in the following three cases. The runoff length for each case should be determined using Equation 3-24.

3.3.8.8.1 Case I

The whole of the traveled way, including the median, is superelevated as a plane section. Case I should be limited to narrow medians and moderate superelevation rates to avoid substantial differences in elevation of the extreme edges of the traveled way arising from the median tilt. Specifically, Case I should be applied only to medians with widths of 15 ft [4 m] or less. Superelevation can be attained using a method similar to that shown in Figure 3-8A except for the two median edges, which will appear as profiles only slightly removed from the centerline. For Case I designs, the length of runoff should be based on the total rotated width (including the median width). However, because narrow medians have very little effect on the runoff length, medians widths of up to 10 ft [3 m] may be ignored when determining the runoff length.

3.3.8.8.2 Case II

The median is held in a horizontal plane and the two traveled ways are rotated separately around the median edges. Case II can be applied to any width of median but is most appropriate for medians with widths between 15 and 60 ft [4 and 18 m]. By holding the median edges level, the difference in elevation between the extreme traveled-way edges can be limited to that needed to superelevate the roadway. Superelevation transition designs for Case II usually have the roadways rotated about the median-edge of pavement. Superelevation can be attained using any of the methods shown in Figures 3-8B, 3-8C, and 3-8D, with the profile reference line being the same for both traveled ways. Where Case II is used for a narrow median width of 10 ft [3 m] or less held in a horizontal plane, the runoff lengths may be the same as those for a single undivided highway.

3.3.8.8.3 Case III

The two traveled ways are treated separately for runoff which results in variable differences in elevations at the median edges. Case III design can be used with wide medians (i.e., median widths of 60 ft [18 m] or more). For this case, the differences in elevation of the extreme edges of the traveled way are minimized by a compensating slope across the median. With a wide median, the profiles and superelevation transition may be designed separately for the two roadways. Accordingly, superelevation can be attained by the method otherwise considered appropriate (i.e., any of the methods in Figure 3-8 can be used).

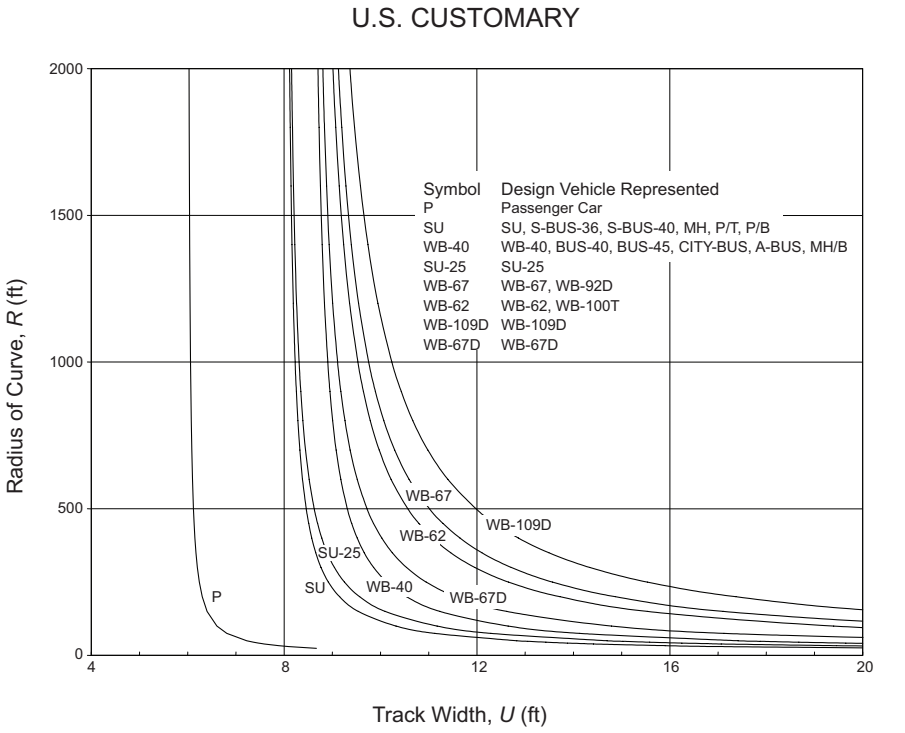
The lateral clearance allowance, C, provides clearance between the edge of the traveled way and nearest wheel path and for the body clearance between vehicles passing or meeting. Lateral clearance per vehicle is assumed to be 2.0, 2.5, and 3.0 ft [0.6, 0.75, and 0.9 m] for tangent two-lane traveled way widths, Wn, equal to 20, 22, and 24 ft [6.0, 6.6, and 7.2 m], respectively.

The width of the front overhang (FA) is the radial distance between the outer edge of the tire path of the outer front wheel and the path of the outer front edge of the vehicle body. For curves and turning roadways, FA depends on the radius of the curve, the extent of the front overhang of the design vehicle, and the wheelbase of the unit itself. In the case of tractor-trailer combinations, only the wheelbase of the tractor unit is used. Figure 3-10 illustrates relative overhang width values for FA determined from:

U.S. Customary	Metric
$F_A = \sqrt{R^2 + A(2L + A)} - R$	$F_A = \sqrt{R^2 + A(2L + A)} - R$
where:	where:
F_A = width of front overhang, ft	F_A = width of front overhang, m
R = radius of curve or turning roadway (two-lane), ft	R = radius of curve or turning roadway (two-lane), m
A = front overhang of inner lane vehicle, ft	A = front overhang of inner lane vehicle, m
L = wheelbase of single unit or tractor, ft	L = wheelbase of single unit or tractor, m

(3-33)

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.



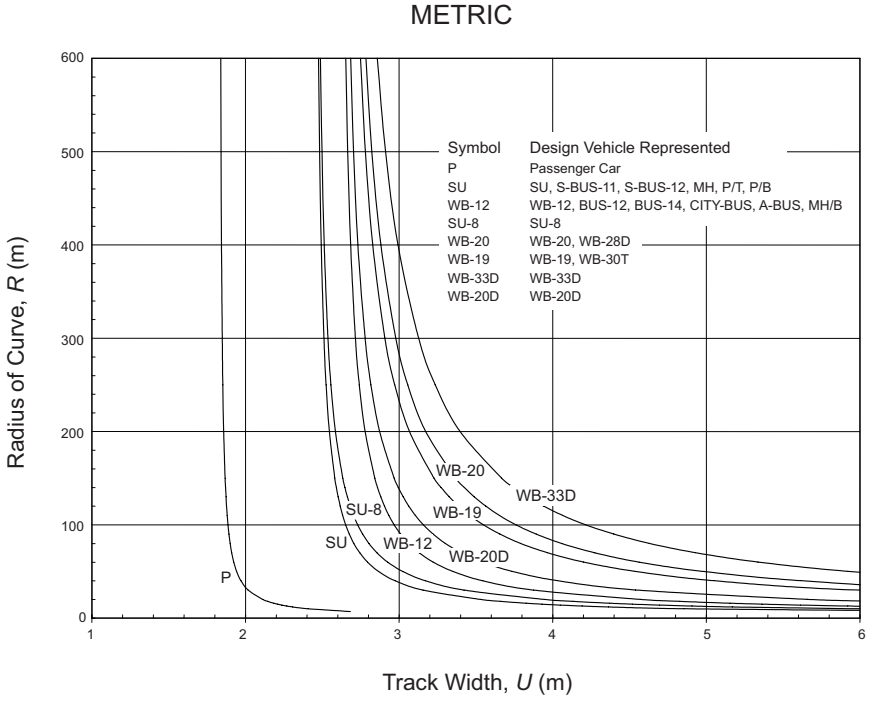


Figure 3-9. Track Width for Widening of Traveled Way on Curves

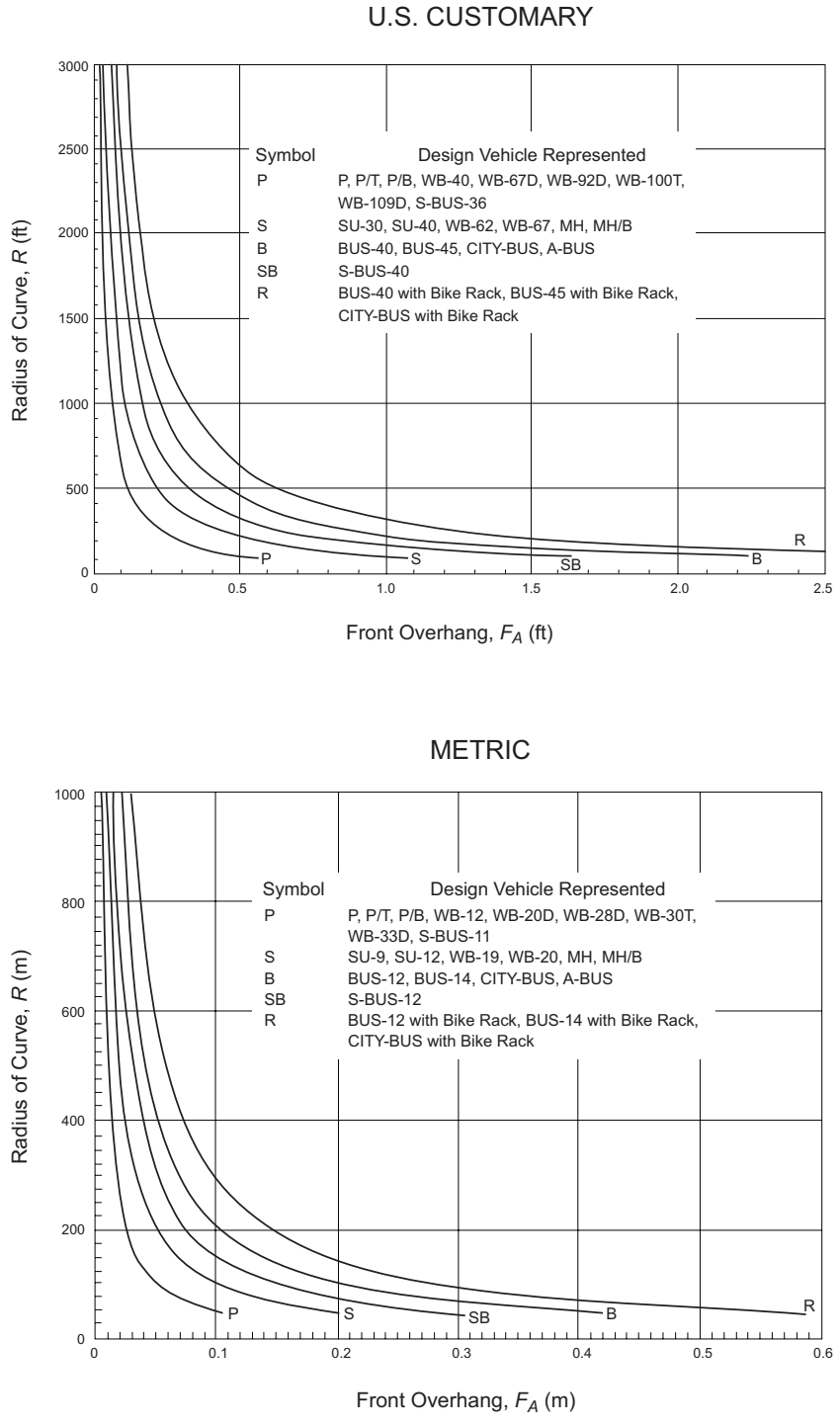


Figure 3-10. Front Overhang for Widening of Traveled Way on Curves

The width of the rear overhang (FB) is the radial distance between the outer edge of the tire path of the inner rear wheel and the inside edge of the vehicle body. For the passenger car (P) design vehicle, the width of the body is 1 ft [0.3 m] greater than the width of out-to-out width

of the rear wheels, making FB = 0.5 ft [0.15 m]. In the truck design vehicles, the width of body is the same as the width out-to-out of the rear wheels, and FB = 0.

The extra width allowance (Z) is an additional radial width of pavement to accommodate the difficulty of maneuvering on a curve and the variation in driver operation. This additional width is an empirical value that varies with the speed of traffic and the radius of the curve. The additional width allowance is expressed as:

U.S. Customary	Metric
$Z = \left(V / \sqrt{R}\right)$	$Z = 0.1\left(V / \sqrt{R}\right)$ (3-34)
where:	where:
Z = extra width allowance, ft	Z = extra width allowance, m
V = design speed of the highway, mph	V = design speed of the highway, km/h
R = radius of curve or turning roadway (two-lane), ft	R = radius of curve or turning roadway (two-lane), m

This expression, used primarily for widening of the traveled way on open highways, is also applicable to intersection curves. For the normal range of curve radii at intersections, the extra width allowance, Z converges to a nearly constant value of 2 ft [0.6 m] by using the speed–curvature relations for radii in the range of 50 to 500 ft [15 to 150 m]. This added width, as shown diagrammatically in Figures 3-12 and 3-13, should be assumed to be evenly distributed over the traveled way width to allow for the inaccuracy in steering on curved paths.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

values for the WB-62 [WB-19] truck should be adjusted in accordance with Table 3-25. The suggested increases of the tabular values for the ranges of radius of curvature are general and will not necessarily result in a full lateral clearance C or an extra width allowance Z. With the lower speeds and volumes on roads with such curvature, however, slightly smaller clearances may be appropriate.

Table 3-24a. Calculated and Design Values for Traveled Way Widening on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)

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

Notes:

Values shown are for WB-62 design vehicle and represent widening in feet. For other design vehicles, use adjustments in Table 3-25.

Values less than 2.0 ft may be disregarded.

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

For 4-lane roadways, multiply above values by 2.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Table 3-24b. Calculated and Design Values For Traveled Way Widening on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)

Metric																		
Radius of Curve (m)	Traveled way width = 7.2 m						Traveled way width = 6.6 m						Traveled way width = 6.0 m					
	Design Speed (km/h)						Design Speed (km/h)						Design Speed (km/h)					
	50	60	70	80	90	100	50	60	70	80	90	100	50	60	70	80	90	100
3000	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.3	0.3	0.3	0.3	0.3	0.5	0.6	0.6	0.6	0.6	0.6
2500	0.0	0.0	0.0	0.0	0.0	0.1	0.3	0.3	0.3	0.3	0.3	0.4	0.6	0.6	0.6	0.6	0.6	0.7
2000	0.0	0.0	0.0	0.1	0.1	0.1	0.3	0.3	0.3	0.4	0.4	0.4	0.6	0.6	0.6	0.7	0.7	0.7
1500	0.0	0.1	0.1	0.1	0.1	0.2	0.3	0.4	0.4	0.4	0.4	0.5	0.6	0.7	0.7	0.7	0.7	0.8
1000	0.1	0.2	0.2	0.2	0.3	0.3	0.4	0.5	0.5	0.5	0.6	0.6	0.7	0.8	0.8	0.8	0.9	0.9
900	0.2	0.2	0.2	0.3	0.3	0.3	0.5	0.5	0.5	0.6	0.6	0.6	0.8	0.8	0.8	0.9	0.9	0.9
800	0.2	0.2	0.3	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.8	0.8	0.9	0.9	0.9	1.0
700	0.3	0.3	0.3	0.4	0.4	0.4	0.6	0.6	0.6	0.7	0.7	0.7	0.9	0.9	0.9	1.0	1.0	1.0
600	0.3	0.4	0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8	0.8	0.9	1.0	1.0	1.0	1.1	1.1
500	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2	1.2
400	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2	1.2	1.3	1.3	1.4
300	0.7	0.8	0.8	0.9	1.0	1.0	1.0	1.1	1.1	1.2	1.3	1.3	1.3	1.4	1.4	1.5	1.6	1.6
250	0.9	1.0	1.0	1.1	1.1		1.2	1.3	1.3	1.4	1.4		1.5	1.6	1.6	1.7	1.7	
200	1.1	1.2	1.3	1.3			1.4	1.5	1.6	1.6			1.7	1.8	1.9	1.9		
150	1.5	1.6	1.7	1.8			1.8	1.9	2.0	2.1			2.1	2.2	2.3	2.4		
140	1.6	1.7					1.9	2.0					2.2	2.3				
130	1.8	1.8					2.1	2.1					2.4	2.4				
120	1.9	2.0					2.2	2.3					2.5	2.6				
110	2.1	2.2					2.4	2.5					2.7	2.8				
100	2.3	2.4					2.6	2.7					2.9	3.0				
90	2.5						2.8						3.1					
80	2.8						3.1						3.4					
70	3.2						3.5						3.8					

Notes:

Values shown are for WB-19 design vehicle and represent widening in meters. For other design vehicles, use adjustments in Table 3-25.

Values less than 0.6 m may be disregarded.

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

For 4-lane roadways, multiply above values by 2.

passenger cars. Trucks generally increase speed by up to 5 percent on downgrades and decrease speed by 7 percent or more on upgrades as compared to their operation on level terrains. On upgrades, the maximum speed that can be maintained by a truck is dependent primarily on the length and steepness of the grade and the truck's weight/power ratio, which is the gross vehicle weight divided by the net engine power. Other factors that affect the average truck speed on a grade are the entering speed, the aerodynamic resistance, and skill of the driver. The last two factors cause only minor variations in the average speed on grade.

Extensive studies of truck performance have been conducted to determine the separate and combined effects of roadway grade, tractive effort, and gross vehicle weight (*20, 27, 38, 39, 54, 64, 77*). Truck engines have become more powerful, relative to the loads transported, continuously for many years. The average weight/power ratio for heavy trucks decreased from 360 lb/hp [220 kg/kW] in 1949 to 200 lb/hp [120 kg/kW] in 2000 (*33*). A weight/power ratio of 140 lb/hp [85 kg/kW] is more representative of the trucks on the road today (*66*).

The effect of rate and length of grade on the speed of a typical heavy truck with a weight/power ratio of 140 lb/hp [85 kg/kW] is shown in Figures 3-16 and 3-17. From Figure 3-15 it can be determined how far a truck, starting its climb from any speed up to approximately 70 mph [110 km/h], travels up various grades or combinations of grades before a certain or uniform speed is reached. For instance, with an entering speed of approximately 70 mph [110 km/h], a truck travels about 5,000 ft [1,500 m] up a 6 percent grade before its speed is reduced to 35 mph [60 km/h]. If the entering speed is 50 mph [80 km/h], the speed at the end of a 3,000-ft [900-m] climb is about 35 mph [60 km/h]. This is determined by starting on the curve for a 6 percent grade corresponding to 50 mph [80 km/h] for which the distance is 2,000 ft [600 m], and proceeding along it to the point where the distance is 3,000 ft [900 m] more, or 5,000 ft [1,500 m], for which the speed is about 35 mph [60 km/h]. Figure 3-16 shows the performance on grade for a truck that approaches the grade at or below crawl speed. The truck is able to accelerate to a speed of 30 mph [50 km/h] or more only on grades of 6 percent or less. Trucks with weight/power ratios of 140 lb/hp [85 kg/kW] should be able to maintain a minimum speed of 50 mph [80 km/h] on a 2 percent upgrade. These data serve as a valuable guide for design in appraising the effect of trucks on traffic operation for a given set of profile conditions.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

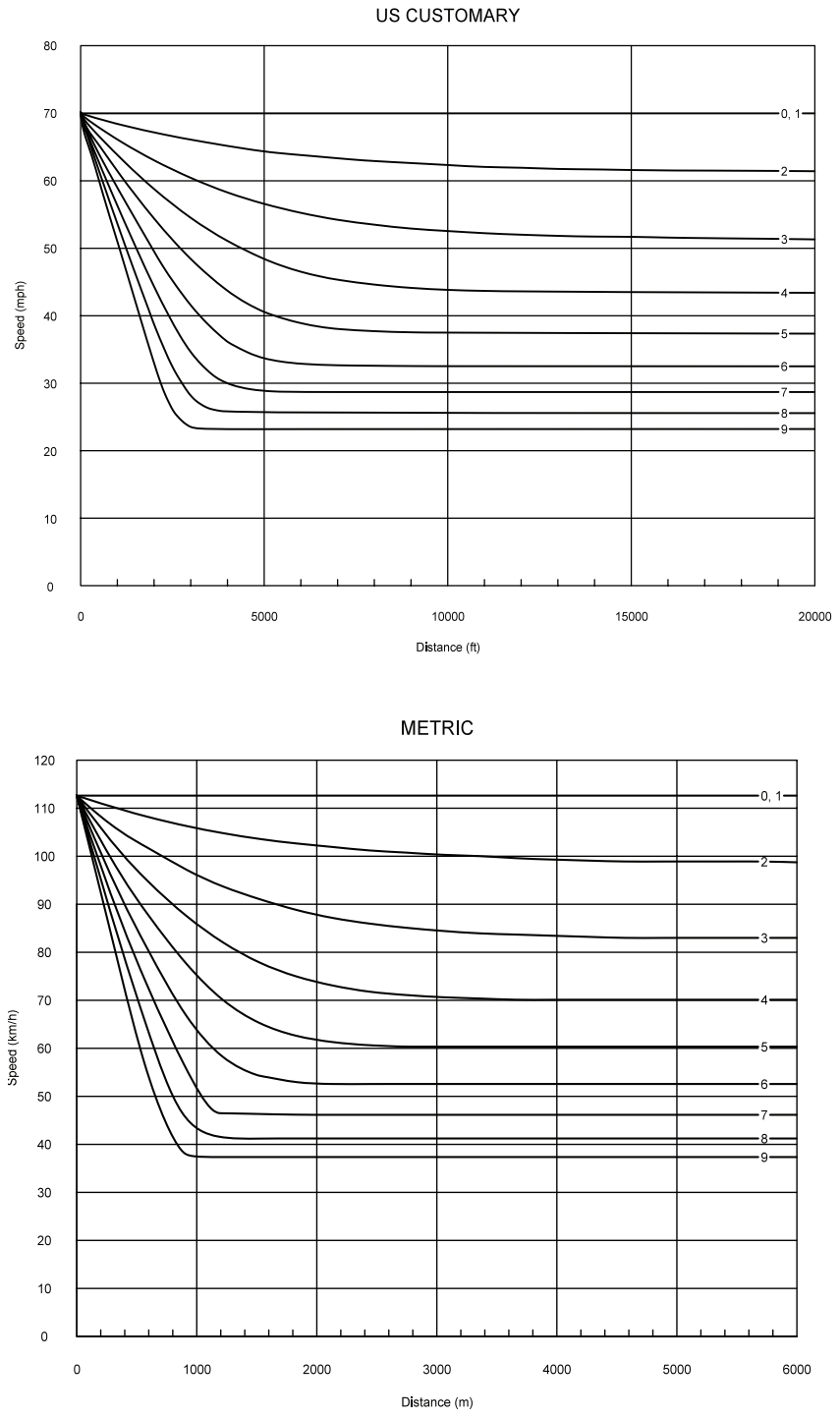


Figure 3-15. Speed–Distance Curves for a Typical Heavy Truck of 140 lb/hp [85 kg/kW] for Deceleration on Upgrades

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

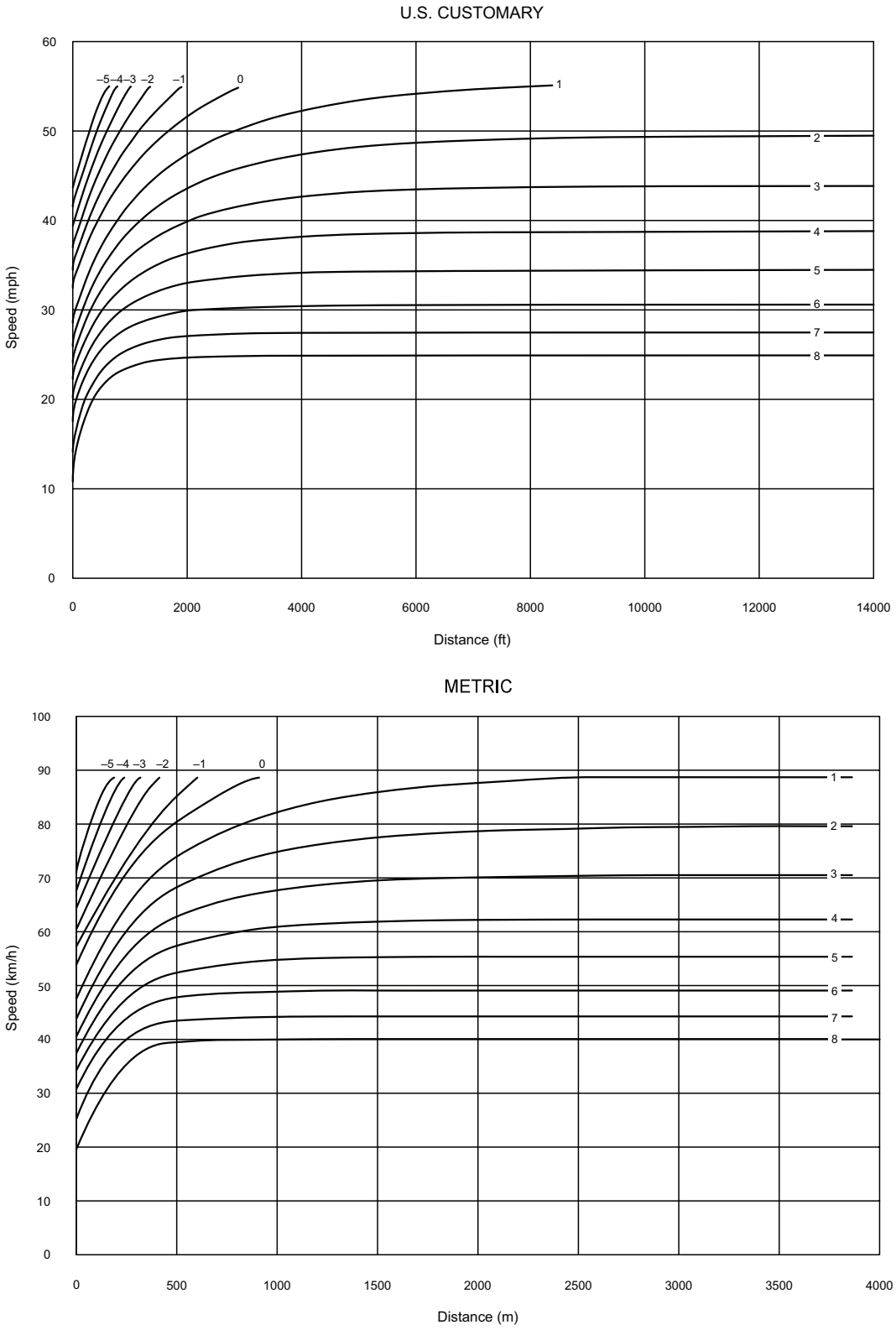


Figure 3-16. Speed–Distance Curves for Acceleration of a Typical Heavy Truck of 140 lb/hp [85 kg/kW] on Upgrades and Downgrades

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Taking all factors into account, it appears conservative to use a weight/power ratio of 140 lb/hp [85 kg/kW] in determining critical length of grade, as presented in Figures 3-15 and 3-16. In some states, larger and heavier trucks similar to the WB-92D [WB-28D], WB-100T [WB-30T], and WB-109D [WB-33D] design vehicles are allowed. Where such trucks present in sufficient volumes to serve as the design vehicle, consideration may be given to using a truck with a weight/power ratio of 200 lb/hp [120 kg/kW] in determining critical length of grade, as shown in Figures 3-17 and 3-18.

3.4.2.1.3 Recreational Vehicles

Consideration of recreational vehicles on grades is not as critical as consideration of trucks. However, on certain routes such as designated recreational routes, where a low percentage of trucks may not warrant a truck climbing lane, sufficient recreational vehicle traffic may indicate a need for an additional lane. This can be evaluated by using the design charts in Figure 3-19 in the same manner as for trucks described in Section 3.4.2.1.2. Recreational vehicles include self-contained motor homes, pickup campers, and towed trailers of numerous sizes. Because the characteristics of recreational vehicles vary so much, it is difficult to establish a single design vehicle. However, one study on the speed of vehicles on grades included recreational vehicles (*75*). The critical vehicle was considered to be a vehicle pulling a travel trailer, and the charts in Figure 3-19 for a typical recreational vehicle are based on that assumption.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

3. One of the following conditions exists:
   - A 10-mph [15-km/h] or greater speed reduction is expected for a typical heavy truck.
   - Level of service E or F exists on the grade.
   - A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.

In addition, high crash frequencies may justify the addition of a climbing lane regardless of grade or traffic volumes.

The upgrade flow rate is determined by multiplying the predicted or existing design hour volume by the directional distribution factor for the upgrade direction and dividing the result by the peak hour factor (the peak hour and directional distribution factors are discussed in Section 2.3). The number of upgrade trucks is obtained by multiplying the upgrade flow rate by the percentage of trucks in the upgrade direction.

3.4.3.1.2 Trucks

As indicated in the immediately preceding paragraphs, only one of the three conditions specified in Criterion 3 need be met. The critical length of grade to effect a 10-mph [15-km/h] speed reduction for trucks is found using Figure 3-21. This critical length is compared with the length of the particular grade being evaluated. If the critical length of grade is less than the length of the grade being studied, Criterion 3 is satisfied. This evaluation should be done first because, where the critical length of grade is exceeded, no further evaluations under Criterion 3 will be needed.

Justification for climbing lanes where the critical length of grade is not exceeded should be considered from the standpoint of highway capacity. The procedures used are those from the HCM (*67*) for analysis of specific grades on two-lane highways. The remaining conditions in Criterion 3 are evaluated using these HCM procedures. The effect of trucks on capacity is primarily a function of the difference between the average speed of the trucks and the average running speed of the passenger cars on the highway. Physical dimensions of heavy trucks and their poorer acceleration characteristics also have a bearing on the space they need in the traffic stream.

On individual grades the effect of trucks is more severe than their average effect over a longer section of highway. Thus, for a given volume of mixed traffic and a fixed roadway cross section, a higher degree of congestion is experienced on individual grades than for the average operation over longer sections that include downgrades as well as upgrades. To determine the design service volume on individual grades, use truck factors derived from the geometrics of the grade and the level of service selected by the highway agency as the basis for design of the highway under consideration.

If there is no 10-mph [15-km/h] reduction in speed (i.e., if the critical length of grade is not exceeded), the level of service on the grade should be examined to determine if level of service E or F exists. This is done by calculating the limiting service flow rate for level of service D and comparing this rate to the actual flow rate on the grade. The actual flow rate is determined by dividing the hourly volume of traffic by the peak hour factor. If the actual flow rate exceeds the service flow rate at level of service D, Criterion 3 is satisfied. When the actual flow rate is less than the limiting value, a climbing lane is not warranted by this second element of Criterion 3.

The remaining issue to examine if neither of the other elements of Criterion 3 are satisfied is whether there is a two-level reduction in the level of service between the approach and the upgrade. To evaluate this criterion, the level of service for the grade and the approach segment should both be determined. Since this criterion needs consideration in only a very limited number of cases, it is not discussed in detail here.

The HCM (*67*) provides additional details and worksheets to perform the computations needed for analysis in the preceding criteria. This procedure is also available in computer software, reducing the need for manual calculations.

Because there are so many variables involved, virtually no given set of conditions can be properly described as typical. Therefore, a detailed analysis such as the one described is recommended wherever climbing lanes are being considered.

The location where an added lane should begin depends on the speeds at which trucks approach the grade and on the extent of sight distance restrictions on the approach. Where there are no sight distance restrictions or other conditions that limit speeds on the approach, the added lane may be introduced on the upgrade beyond its beginning because the speed of trucks will not be reduced beyond the level tolerable to following drivers until they have traveled some distance up the grade. This optimum point for capacity would occur for a reduction in truck speed to 40 mph [60 km/h], but a 10-mph [15-km/h] decrease in truck speed below the average running speed, as discussed in Section 3.4.2.3, "Critical Lengths of Grade for Design," is the most practical reduction obtainable from the standpoint of level of service and crash frequency. This 10-mph [15-km/h] reduction is the accepted basis for determining the location at which to begin climbing lanes. The distance from the bottom of the grade to the point where truck speeds fall to 10 mph [15 km/h] below the average running speed may be determined from Figures 3-17 or 3-21. Different curves would apply for trucks with other than a weight/power ratio of 200 lb/hp [120 kg/kW]. For example, assuming an approach condition on which trucks with a 200-lb/hp [120-kg/kW] weight/power ratio are traveling within a flow having an average running speed of 70 mph [110 km/h], the resulting 10-mph [15-km/h] speed reduction occurs at distances of approximately 600 to 1,200 ft [175 to 350 m] for grades varying from 7 to 4 percent. With a downgrade approach, these distances would be longer and, with an upgrade approach, they would be shorter. Distances thus determined may be used to establish the point at which a climbing lane should begin. Where restrictions, upgrade approaches, or other conditions indicate the likeli-

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

order of study cannot be stated for all highways, a general procedure applicable to most facilities is described in this section.

The designer should use working drawings of a size, scale, and arrangement so that he or she can study long, continuous stretches of highway in both plan and profile and visualize the whole in three dimensions. Working drawings should be of a small scale, with the profile plotted jointly with the plan. A continuous roll of plan–profile paper usually is suitable for this purpose. To assist in this visualization, there also are programs available for personal computers (PCs) that allow designers to view proposed vertical and horizontal alignments in three dimensions.

After study of the horizontal alignment and profile in preliminary form, adjustments in either, or both, can be made jointly to obtain the desired coordination. At this stage, the designer should not be concerned with line calculations other than known major controls. The study should be made largely on the basis of a graphical or computer analysis. The criteria and elements of design covered in this and the preceding chapter should be kept in mind. For the selected design speed, the values for controlling curvature, gradient, sight distance, and superelevation runoff length should be obtained and checked graphically or with a computer or CADD system. Design speed may have to be adjusted during the process along some sections to conform to likely variations in speeds of operation. This need may occur where noticeable changes in alignment characteristics are needed to accommodate unusual terrain or right-of-way controls. In addition, the general design controls, as enumerated separately for horizontal alignment, vertical alignment, and their combination, should be considered. All aspects of terrain, traffic operations for all transportation modes, and appearance should be considered and the horizontal and vertical lines should be adjusted and coordinated before the costly and time-consuming calculations and the preparation of construction plans to large scale are started.

The coordination of horizontal alignment and profile from the standpoint of appearance usually can be accomplished visually on the preliminary working drawings or with the assistance of computer programs that have been developed for this purpose. Generally, such methods result in a satisfactory product when applied by an experienced designer. This means of analysis may be supplemented by models, sketches, or images projected by a computer at locations where the appearance of certain combinations of line and grade is unclear. For highways with gutters, the effects of superelevation transitions on gutter-line profiles should be examined. This can be particularly significant where flat grades are involved and can result in local depressions. Slight shifts in profile in relation to horizontal curves can sometimes eliminate this concern.

The procedures described above should obviously be modified for the design of typical local roads or streets, as compared to higher type highways. The alignment of any local road or street, whether for a new roadway or for reconstruction of an existing roadway, is governed by the existing or likely future development along it. Where driveways are located on or near a horizontal curve or crest vertical curve, the designer should check the availability of adequate sight distance for major-road drivers approaching from the rear of a stopped or turning vehicle and

for major-road drivers turning left from the major road into the driveway. In addition, the availability of sight distance for left turns from divided highways should be checked because of the possibility of sight obstructions in the median. The horizontal and vertical alignment of intersecting roadways at intersections and driveways are key controls. Although they should be fully considered, they should not override the broader desirable features described above. Even for street design, it is desirable to work out long, flowing alignment and profile sections rather than a connected series of block-by-block sections. Some examples of poor and good practice are illustrated in Figure 3-40.

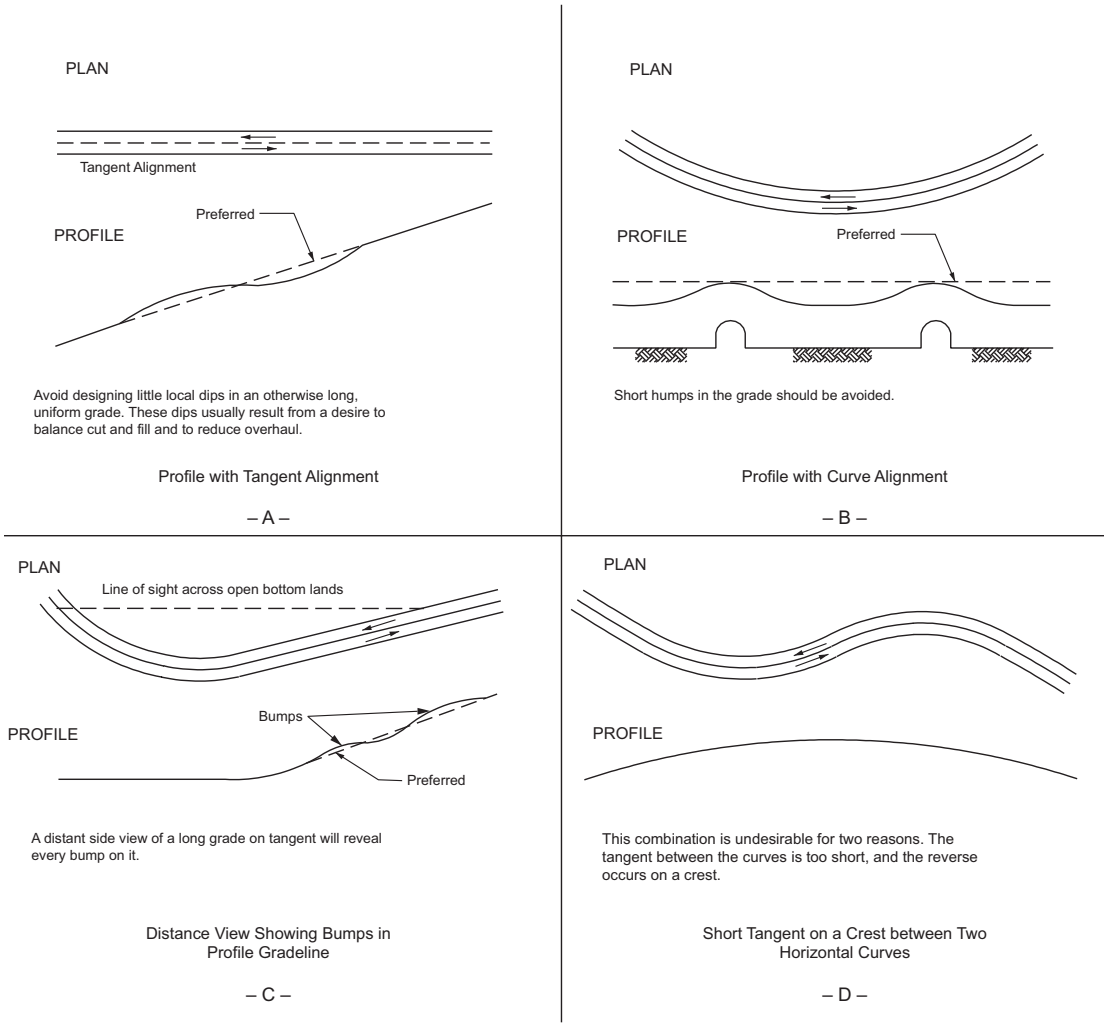


Figure 3-40. Alignment and Profile Relationships in Roadway Design (*43*)

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

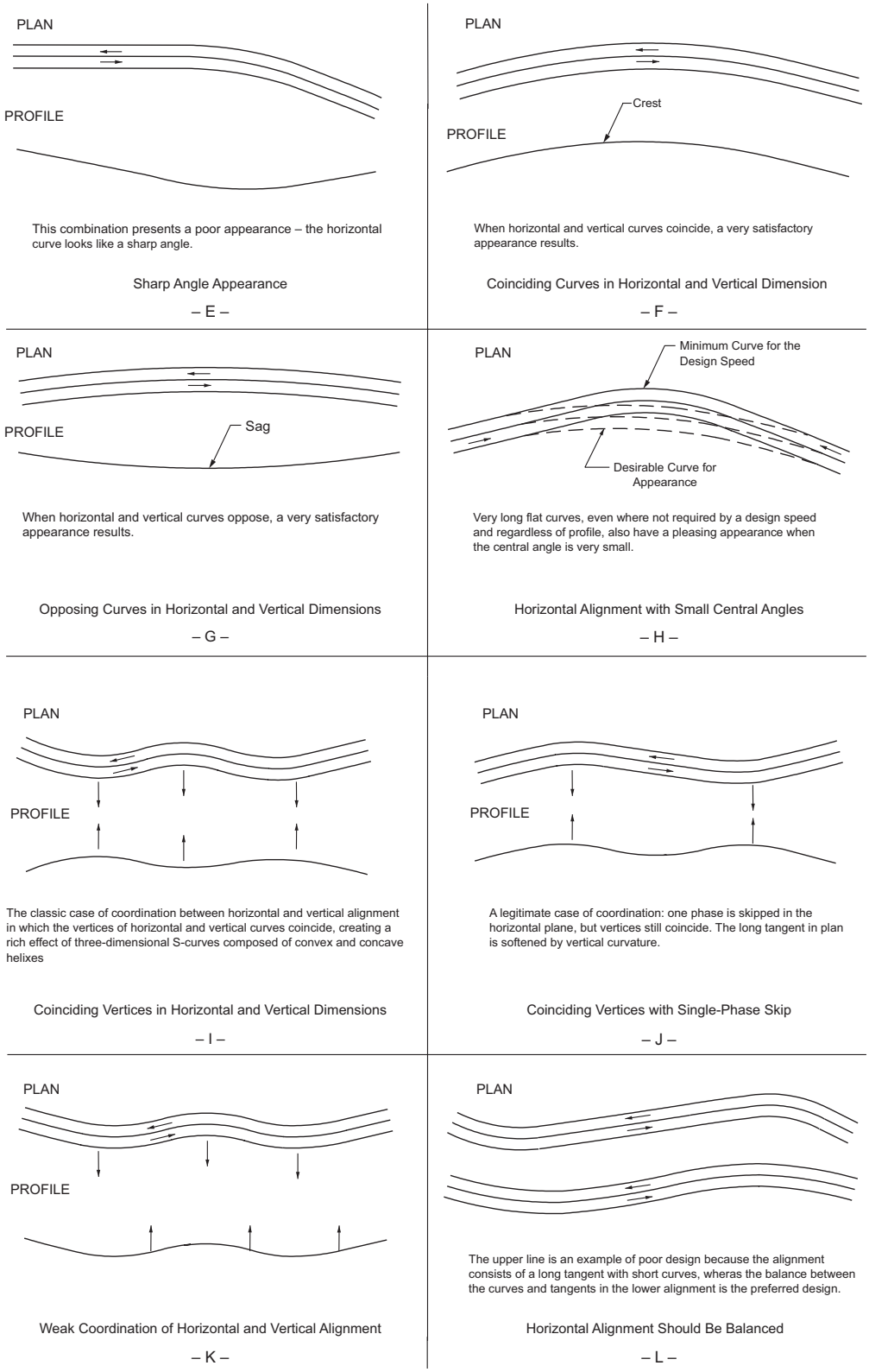


Figure 3-40. Alignment and Profile Relationships in Roadway Design (Continued)

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

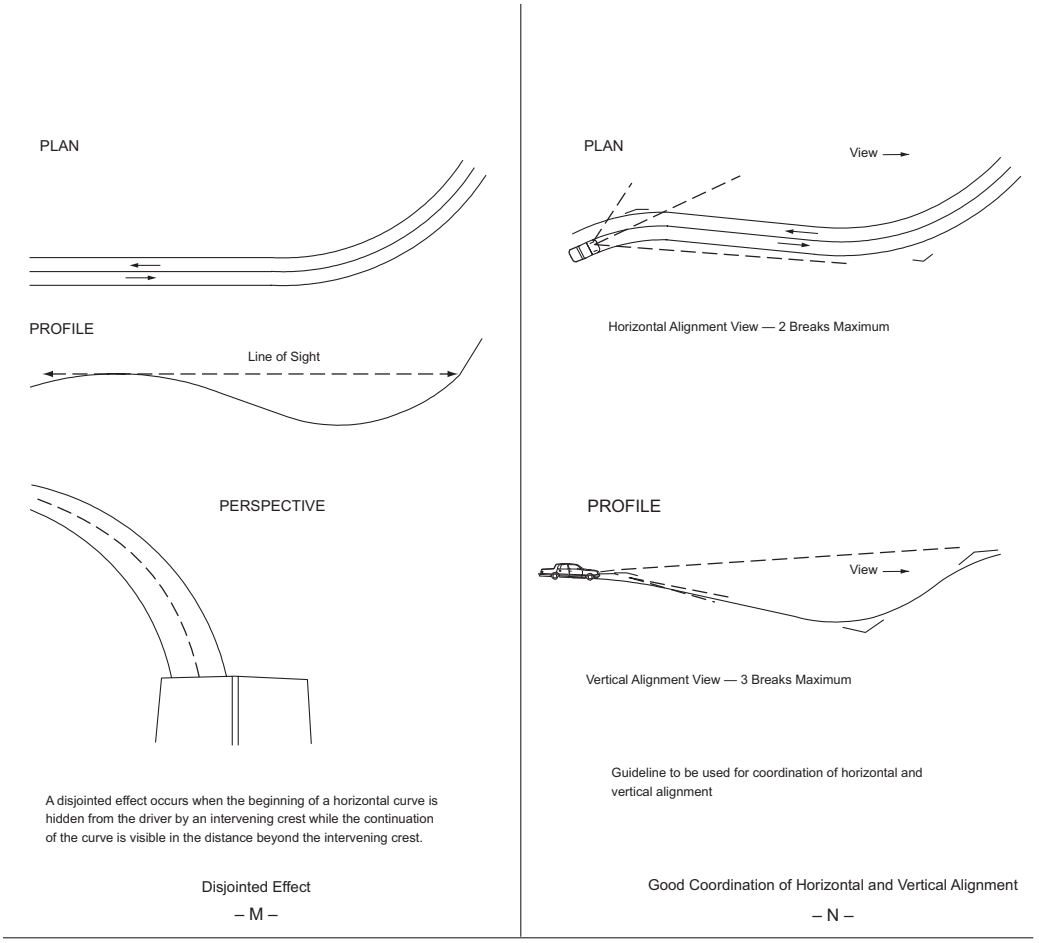


Figure 3-40. Alignment and Profile Relationships in Roadway Design (Continued)

3.6 OTHER FEATURES AFFECTING GEOMETRIC DESIGN

In addition to the design elements discussed previously, several other features affect or are affected by the geometric design of a roadway. Each of these features is discussed only to the extent needed to show its relation to geometric design and how it, in turn, is thereby affected. Detailed design of these features is not covered here.

3.6.1 Erosion Control and Landscape Development

Erosion prevention is one of the major factors in design, construction, and maintenance of highways. It should be considered early in the location and design stages. Some degree of erosion control can be incorporated into the geometric design, particularly in the cross section elements. Of course, the most direct application of erosion control occurs in drainage design and in the writing of specifications for landscaping and slope planting.

12. AASHTO. *AASHTO LRFD Bridge Design Specifications*, Eighth Edition with 2018 Errata, LRFD-8. American Association of State Highway and Transportation Officials, Washington, DC, 2017.

13. AASHTO. *LRFD Road Tunnel Design and Construction Guide Specifications*, First Edition, LRFDTUN-1. American Association of State Highway and Transportation Officials, Washington, DC, 2017.

14. Anderson, D. A., J. R. Reed, R. S. Huebner, J. J. Henry, W. P. Kilareski, and J. C. Warner. *Improved Surface Drainage of Pavements.* NCHRP Project I-29, Pennsylvania Transportation Institute, Pennsylvania State University, Federal Highway Administration, Washington, DC, 1998, p. 42. Available at

https://www.nap.edu/catalog/6357/improved-surface-drainage-of-pavements-final-report

15. Barry, T. M. and J. A. Reagan. *FHWA Highway Traffic Noise Prediction Model.* FHWA-RD-77-108. Federal Highway Administration, U.S. Department of Transportation, McLean, VA, 1978.

16. Bonneson, J. A. and P. T. McCoy. *National Cooperative Highway Research Program Report 395: Capacity and Operational Effects of Midblock Left-Turn Lanes.* NCHRP, Transportation Research Board, Washington, DC, 1997. Available at

http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_395.pdf

17. Brewer, Marcus A. *National Cooperative Highway Research Program Synthesis 432: Recent Roadway Geometric Design Research for Improved Safety and Operations.* NCHRP, Transportation Research Board, Washington, DC, 2012.

18. Brown, S. A., S. M. Stein, and J. C. Warner. *Hydraulic Engineering Circular No. 22* (*HEC 22*)*, Urban Drainage Design Manual, 2009 Edition.* FHWA-NHI-10-009. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2009, p. 4-4. Available at

https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf

19. Bucko, T. R. and A. Khorashadi. *Evaluation of Milled-In Rumble Strips, Rolled-In Strips and Proprietary Applications.* California Department of Transportation, Sacramento, CA, May 2001.

20. Downs, Jr., H. G., and D. W. Wallace. *National Cooperative Highway Research Program Report 254: Shoulder Geometrics and Use Guidelines.* NCHRP, Transportation Research Board, Washington, DC, 1982.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

21. Dunlap, D. F., P. S. Fancher, R. E. Scott, C. C. MacAdam, and L. Segal. *National Cooperative Highway Research Program Report 184: Influence of Combined Highway Grade and Horizontal Alignment on Skidding.* NCHRP, Transportation Research Board, Washington, DC, 1978.

22. FHWA. Computer software and related publications are available from McTrans, 512 Weil Hall, University of Florida, Gainesville, Florida 32611-2083. Telephone (904) 392-0378 or PC-TRANS, 2011 Learned Hall, University of Kansas, Lawrence, Kansas, 66045. Telephone (913) 864-3199. Available at

    http://mctrans.ce.ufl.edu

    HY 22. Urban Drainage Design Programs, Version 2.1, 1998.

    HY 7. Bridge Waterways Analysis Model, (WSPRO), 2005.

    FHWA-SA-98-080. WSPRO Users Manual (Version 061698), 1998.

    HY 8. FHWA Culvert Analysis (Version 7.1), 2008. Research Report (Version 1.0), 1987.

    HYDRAIN. Drainage Design System (Version 6.1), 1999.

    HYDRAIN. Users Manual, 1999.

23. FHWA. *Design of Roadside Channels with Flexible Linings*, 3rd Edition. Hydraulic Engineering Circular (HEC-15). FHWA NHI-05-114. Office of Engineering, Bridge Division, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2005. Available at

    http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf

24. FHWA. *Designing Sidewalks and Trails for Access, Part I: Review of Existing Guidelines and Practices.* FHWA-HEP-99-006. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, July 1999. Available at

    https://www.fhwa.dot.gov/environment/bicycle_pedestrian/publications/sidewalks/index.cfm

25. FHWA. *Designing Sidewalks and Trails for Access, Part II: Best Practices Design Guide.* FHWA-EP-01-027. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, September 2001. Available at

    https://www.fhwa.dot.gov/environment/bicycle_pedestrian/publications/sidewalk2/pdf.cfm

26. FHWA. *Highway Traffic Noise: Analysis and Abatement Guide*, Report No. FHWA-HEP-10-025, FHWA, Washington, DC, December 2011.

27. FHWA. *Manual on Uniform Traffic Control Devices for Streets and Highways*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2009. Available at

http://mutcd.fhwa.dot.gov

28. FHWA. *Rumble Strips and Rumble Stripes.* Federal Highway Administration, U.S. Department of Transportation, Washington, DC, December 2016. Available at

http://safety.fhwa.dot.gov/roadway_dept/pavement/rumble_strips/

29. FHWA. *Separated Bike Lane Planning and Design Guide*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, May 2015.

30. Gattis, J., J. S. Gluck, J. M. Barlow, R. W. Eck, W. F. Hecker, and H. S. Levinson. *National Cooperative Highway Research Program Report 659: Guide for the Geometric Design of Driveways.* NCHRP, Transportation Research Board, Washington, DC, 2010. Available at

http://www.trb.org/Publications/Blurbs/163868.aspx

31. Griffith, M. S. Safety Evaluation of Rolled-In Continuous Shoulder Rumble Strips Installed on Freeways. In *Transportation Research Record 1665*. TRB, National Research Council, Washington, DC, October 1999, pp. 28–34. Available for purchase at

https://trrjournalonline.trb.org/doi/10.3141/1665-05

32. Harwood, D. W. and C. J. Hoban. *Low Cost Methods for Improving Traffic Operation on Two-Lane Roads.* FHWA IP-87-2. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1987.

33. Harwood, D. W., M. T. Pietrucha, M. D. Wooldridge, R. E. Brydia, and K. Fitzpatrick. *National Cooperative Highway Research Program Report 375: Median Intersection Design.* NCHRP, Transportation Research Board, Washington, DC, 1995.

34. Harwood, D. W., J. M. Hutton, C. Fees, K. M. Bauer, A. Glen, and H. Ouren. *National Cooperative Highway Research Program Report 783: Evaluation of the 13 Controlling Criteria for Geometric Design.* NCHRP, Transportation Research Board, Washington, DC, 2014.

35. Harwood, D. W., D. K. Gilmore, J. L. Graham, and M. K. O'Laughlin. *National Cooperative Highway Research Program Report 790: Factors Contributing to Median Encroachments and Cross-Median Crashes.* NCHRP, Transportation Research Board, Washington, DC, 2014.

36. ITE. *Guidelines for Driveway Design and Location*. Institute of Transportation Engineers, Washington, DC, 1986.

37. Lagasse, P. F., J. D. Schall, and E. V. Richardson. *Stream Stability at Highway Structures*, 3rd Edition. Hydraulic Engineering Circular (HEC-20). FHWA-NHI-01-002. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, March 2001. Available at

http://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=19&id=43

38. Marquis, E. L., R. M. Olson, and G. D. Weaver. *National Cooperative Highway Research Program Report 158: Selection of Safe Roadside Cross Sections*. NCHRP, Transportation Research Board, Washington, DC, 1975.

39. Richards, D. and D. Ham. *Serrated Soft-Rock Cut Slopes*. FHWA-RDDP-5-1. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1973.

40. Simpson, M. A. *Noise Barrier Design Handbook*. FHWA-RD-76-58. Federal Highway Administration, U.S. Department of Transportation, McLean, VA, 1976.

41. TCRP. *Transit Cooperative Research Program Report 19: Guidelines for the Location and Design of Bus Stops*. TCRP, Transportation Research Board, Washington, DC, 1996. Available at

http://onlinepubs.trb.org/onlinepubs/tcrp/tcrp_rpt_19-a.pdf

42. Texas Transportation Institute, Parsons Brinckerhoff Quade and Douglas, Inc., and Pacific Rim Resources, Inc. *National Cooperative Highway Research Program 414: HOV Systems Manual*, NCHRP, TRB, Washington, DC, 1998.

43. Torbic, D. J., J. M. Hutton, C. D. Bokenkroger, K. M. Bauer, D. W. Harwood, D. K. Gilmore, J. E. Dunn, J. R. Ronchetto, E. T. Donnell, H. J. Sommer III, P. Garvey, B. Persaud, and C. Lyon. *National Cooperative Highway Research Program Report 641: Guidance for the Design and Application of Shoulder and Centerline Rumble Strips*. NCHRP, Transportation Research Board, Washington, DC, 2009. Available at

http://www.trb.org/Publications/Blurbs/162610.aspx

44. Transportation Research Board. *Access Management Manual*. Transportation Research Board, Washington, DC, 2015.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

In many instances, resource recovery roads are ultimately used for other (e.g., recreational) purposes. In instances such as these, the original design should take into account all the possible ultimate usages.

5.6 LOW-VOLUME ROADS

A low-volume local road is a road that is functionally classified as a local or minor collector road and has a design average daily traffic volume of 2,000 vehicles per day or less. Nearly 80 percent of the roads in the United States can be classified as such. These roads are primarily used by motorists who travel them frequently and are familiar with their geometric design features. The unique characteristics of these roads are generally accepted and anticipated by the drivers using them. Additionally, encounters with others vehicles are infrequent and, statistically, opportunities for multiple-vehicle crashes are unusual. The geometric design of low-volume roads presents a unique challenge because the very low traffic volumes and reduced frequency of crashes make designs normally applied on higher volume roads less cost-effective.

The AASHTO *Guidelines for Geometric Design of Very Low-Volume Local Roads* (*1*) addresses the unique needs of such roads and the geometric designs appropriate to meet those needs. The AASHTO *Guidelines for Geometric Design of Very Low-Volume Local Roads* (*1*) may be used in lieu of this publication when designing local roads that fit the applicable criteria. The AASHTO guidelines for low-volume roads address issues for which appropriate geometric design guidance differs from the policies normally applied to higher volume roads. For any geometric design issues not addressed in the AASHTO guidelines for low-volume roads, design professionals should consult Sections 5.2 and 5.3, and Chapter 6.

5.7 REFERENCES

1. AASHTO. *Guidelines for Geometric Design of Very Low-Volume Local Roads*, First Edition, VLVLR-1. American Association of State Highway and Transportation Officials, Washington, DC, 2001. Second edition pending.

2. AASHTO. *Guide for the Planning, Design, and Operation of Pedestrian Facilities*, First Edition, GPF-1. American Association of State Highway and Transportation Officials, Washington, DC, 2004. Second edition pending.

3. AASHTO. *Guide for Accommodating Utilities within Highway Right-of-Way*, Fourth Edition, GAU-4. American Association of State Highway and Transportation Officials, Washington, DC, 2005.

4. AASHTO. *Roadway Lighting Design Guide*, Sixth Edition with 2010 Errata, GL-6. American Association of State Highway and Transportation Officials, Washington, DC, 2005. Seventh edition pending 2018.

5. AASHTO. *Roadside Design Guide*, Fourth Edition with 2015 Errata, RSDG-4. American Association of State Highway and Transportation Officials, Washington, DC, 2011.

6. AASHTO. *Guide for the Development of Bicycle Facilities*, Fourth Edition with 2017 Errata, GBF-4. American Association of State Highway and Transportation Officials, Washington, DC, 2012.

7. AASHTO. *AASHTO Drainage Manual*, First Edition, ADM-1. American Association of State Highway and Transportation Officials, Washington, DC, 2014.

8. AASHTO. *Guide for Geometric Design of Transit Facilities on Highways and Streets*, First Edition, TVF-1. Association of State Highway and Transportation Officials, Washington, DC, 2014.

9. AASHTO. *AASHTO LRFD Bridge Design Specifications*, Eighth Edition with 2018 Errata, LRFD-8. American Association of State Highway and Transportation Officials, Washington, DC, 2017.

10. Bert, K. E., M. M. Cohn, W. D. Hurst, C. R. Kuykendall, and R. H. Sullivan. *Accommodation of Utility Plants within the Rights-of-Way of Urban Streets and Highways, Manual of Improved Practice.* ASCE Manual No. 14. Joint publication of American Public Works Association, Chicago, and American Society of Civil Engineers, Reston, VA, July 1974.

11. Byrne, J. J., R. L. Nelson, and P. H. Googins. *Logging Road Handbook—The Effect of Road Design on Hauling Costs*. Handbook No. 183. U.S. Forest Service, U.S. Department of Agriculture, Washington, DC, 1960.

12. FHWA. *Manual on Uniform Traffic Control Devices for Streets and Highways*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2009. Available at

    http://mutcd.fhwa.dot.gov

13. FHWA. *Separated Bike Lane Planning and Design Guide*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, May 2015.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

6.4 REFERENCES

1. AASHTO. *Guidelines for Geometric Design of Very Low-Volume Local Roads*, First Edition, VLVLR-1. American Association of State Highway and Transportation Officials, Washington, DC, 2001. Second edition pending.

2. AASHTO. *Guide for the Planning, Design, and Operation of Pedestrian Facilities*, First Edition, GPF-1. American Association of State Highway and Transportation Officials, Washington, DC, 2004. Second edition pending.

3. AASHTO. *Guide for Accommodating Utilities within Highway Right-of-Way*, Fourth Edition, GAU-4. American Association of State Highway and Transportation Officials, Washington, DC, 2005.

4. AASHTO. *Roadway Lighting Design Guide*, Sixth Edition with 2010 Errata, GL-6. American Association of State Highway and Transportation Officials, Washington, DC, 2005. Seventh edition pending 2018.

5. AASHTO. *Roadside Design Guide*, Fourth Edition with 2015 Errata, RSDG-4. American Association of State Highway and Transportation Officials, Washington, DC, 2011.

6. AASHTO. *Guide for the Development of Bicycle Facilities*, Fourth Edition with 2017 Errata, GBF-4. American Association of State Highway and Transportation Officials, Washington, DC, 2012.

7. AASHTO. *AASHTO Drainage Manual*, First Edition, ADM-1. American Association of State Highway and Transportation Officials, Washington, DC, 2014.

8. AASHTO. *Guide for Geometric Design of Transit Facilities on Highways and Streets*, First Edition, TVF-1. Association of State Highway and Transportation Officials, Washington, DC, 2014.

9. AASHTO. *AASHTO LRFD Bridge Design Specifications*, Eighth Edition with 2018 Errata, LRFD-8. American Association of State Highway and Transportation Officials, Washington, DC, 2017.

10. FHWA. *Manual on Uniform Traffic Control Devices for Streets and Highways*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2009. Available at

    http://mutcd.fhwa.dot.gov

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

11. FHWA. Traffic Analysis Tools website. Available at

https://ops.fhwa.dot.gov/trafficanalysistools/index.htm

12. Gibbons, R. B., C. Edwards, B. Williams, and C. K. Anderson. *Informational Report on Lighting Design for Midblock Crosswalks*, FHWA-HRT-08-053. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, April 2008. Available at

http://www.fhwa.dot.gov/publications/research/safety/08053

13. Maryland Department of Transportation. *When Main Street is a State Highway—Blending Function, Beauty and Identity: A Handbook for Communities and Designers*, Maryland State Highway Administration, Maryland Department of Transportation, Baltimore, MD, 2001.

14. Oregon Department of Transportation. *Main Street… When a Highway Runs Through It: A Handbook for Oregon Communities*, Oregon Department of Transportation, Salem, OR, November 1999.

15. Semler, C., A. Vest, K. Kingsley, S. Mah, W. Kittelson, C. Sundstrom, and K. Brookshire. *Guidebook for Developing Pedestrian and Bicycle Performance Measures*, Report No. FHWA-HEP-16-037. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, March 2016.

16. Standard Practice Committee of the IESNA Roadway Lighting Committee. American National *Standard Practice for Roadway Lighting*, ANSI/IESNA RP-8-00. Illuminating Engineering Society of North America, New York, NY, published by ANSI, Washington, DC, 2000.

17. Torbic, D. J., D. K. Gilmore, K. M. Bauer, C. D. Bokenkroger, D. W. Harwood, L. L. Lucas, R. J. Frazier, C. S. Kinzel, D. L. Petree, and M. D. Forsberg. *National Cooperative Highway Research Program Report 737: Design Guidance for High-Speed to Low-Speed Transition Zones for Rural Highways*. NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2012.

18. TRB. *Access Management Manual*. Transportation Research Board, National Research Council, Washington, DC, 2015 or most recent edition.

19. TRB. *Highway Capacity Manual: A Guide for Multimodal Mobility Analysis*, Sixth Edition, HCM6. Transportation Research Board, National Research Council, Washington, DC, 2016.

Report No. FHWA-HEP-17-024. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, December 2016.

12. FHWA. *Manual on Uniform Traffic Control Devices for Streets and Highways*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2009. Available at

    http://mutcd.fhwa.dot.gov

13. FHWA. *Engineering Countermeasure for Reducing Speeds: A Desktop Reference of Potential Effectiveness*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, May 2009.

14. FHWA. *Guidebook for Developing Pedestrian and Bicycle Performance Measures*, Report No. FHWA-HEP-16-037. Federal Highway Administration, U.S. Department of Transportation, March 2016.

15. FHWA. *Achieving Multimodal Networks: Applying Design Flexibility and Reducing Conflicts,* Report No. FHWA-HEP-16-055. Federal Highway Administration, U.S. Department of Transportation, August 2016.

16. Fitzpatrick, K., M. A. Brewer, P. Dorothy, and E. S. Park. *National Cooperative Highway Research Program Report 780: Design Guidance for Intersection Auxiliary Lanes*. NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2014.

17. Gattis, J. L., J. S. Gluck, J. M Barlow, R. W. Eck, W. F. Hecker, and H. S. Levinson. *National Cooperative Highway Research Program Report 659: Guide for the Geometric Design of Driveways*. NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2010.

18. Gibbons, R. B., C. Edwards, B. Williams, and C. K. Anderson. *Informational Report on Lighting Design for Midblock Crosswalks*, Report No. FHWA-HRT-08-053. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, April 2008. Available at

    http://www.fhwa.dot.gov/publications/research/safety/08053

19. Gluck, J., H. S. Levinson, and V. Stover. *National Cooperative Highway Research Program Report 420: Impacts of Access Management Techniques*. NCHRP, Transportation Research Board, National Research Council, Washington, DC, 1999. Available at

    http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_420.pdf

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

20. Graham, J.L., D.W. Harwood, K.R. Richard, M.K. O'Laughlin, E.T. Donnel, and S.N. Brennan, *National Cooperative Highway Research Program Report 794: Median Cross Section Design for Rural Divided Highways.* NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2014.

21. Harwood, D. W., M. T. Pietrucha, M. D. Wooldridge, R. E. Brydia, and K. Fitzpatrick. *National Cooperative Highway Research Program Report 375: Median Intersection Design*. NCHRP, Transportation Research Board, National Research Council, Washington, DC, 1995.

22. Lutkevich, P., D. Mclean, and J. Cheung, *FHWA Lighting Handbook*, Report No. FHWA-SA-11-22. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, August 2012.

23. Maryland Department of Transportation. *When Main Street is a State Highway—Blending Function, Beauty and Identity: A Handbook for Communities and Designers*. Maryland State Highway Administration, Maryland Department of Transportation, Baltimore, MD, 2001.

24. Oregon Department of Transportation. *Main Street… When a Highway Runs Through It: A Handbook for Oregon Communities*. Oregon Department of Transportation, Salem, OR, November 1999.

25. Ryus, P., E. Ferguson, K.M. Lausten, R.J. Schnieder, F.R. Proulx, T. Hull, and L. Miranda-Moreno, *National Cooperative Highway Research Program Web-Only Document 205: Methodologies and Technologies for Pedestrian and Bicycle Volume Data Collection*. NCHRP, Transportation Research Board, Washington, DC, 2014.

26. Ryus, P., A. Butsick, K.M. Lausten, R.J. Schnieder, F.R. Proulx, T. Hull, and L. Miranda-Moreno, *National Cooperative Highway Research Program Web-Only Document 229: Methodologies and Technologies for Pedestrian and Bicycle Volume Data Collection, Phase* 2. NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2016.

27. Ryus, P., E. Ferguson, F.R. Proulx, R.J. Schnieder, and T. Hull, *National Cooperative Highway Research Program Report 797: Guidebook on Pedestrian and Bicycle Volume Data Collection*. NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2014.

28. Semler, C., A. Vest, K. Kingsley, S. Mah, W. Kittelson, C. Sundstrom, and K. Brookshire. *Guidebook for Developing Pedestrian and Bicycle Performance Measures*, Report No. FHWA-

8.2.3 Levels of Service

Procedures for traffic operational analyses for freeways, including appropriate adjustments for operational and highway factors, are presented in the *Highway Capacity Manual* (HCM) (*10*), which also includes a thorough discussion of the level-of-service concept. Designers should strive to provide the highest level of service practical, consistent with anticipated conditions and system constraints. The level of service concept is discussed in Section 2.4.5, and general guidance on customary levels of service for design are summarized in Table 2-3. Freeways and their auxiliary facilities (i.e., ramps, main line weaving sections, and collector–distributor (C–D) roads in the urban and suburban contexts) should generally be designed to provide the highest level of service practical, consistent with a variety of factors including motorist needs, system continuity, community goals, adjacent lane use type and development intensity, social and environmental factors, and aesthetic and historical values.

8.2.4 Traveled Way and Shoulders

Freeways should have a minimum of two through-traffic lanes for each direction of travel. Through-traffic lanes should be 12 ft [3.6 m] wide. Freeway roadways should have a paved surface with adequate skid resistance and structural capacity. Pavement cross slopes should range between 1.5 and 2 percent on tangent sections, with the higher value recommended for areas with moderate rainfall. For areas of heavy rainfall, a pavement cross slope of 2.5 percent may be needed to provide adequate drainage. Appropriate cross-slope rates are discussed in Section 4.2.2. For elevated freeways on viaducts, two-lane pavements usually are sloped to drain the full roadway width toward one side of the roadway. On wider facilities, particularly in areas with heavy rainfall, a crown may be located on the lane line at one-third or one-half the total width from one edge, thus providing two directions for surface drainage. In areas with snowfall, the median and cross slopes of the traveled way should be designed to prevent melting snow stored in the median from draining across the roadway. This is intended to avoid icing conditions during subsequent freezing temperatures.

Guidance for ramp traveled-way widths is presented in Section 10.9.6.

Paved shoulders should be continuous on both the right and left sides of all freeway facilities.

On four-lane freeways, the median (or left) shoulder is normally 4 to 8 ft [1.2 to 2.4 m] wide, at least 4 ft [1.2 m] of which should be paved and the remainder stabilized. The paved width of the right shoulder should be at least 10 ft [3.0 m]; where the DDHV for truck traffic exceeds 250 veh/h, a paved right shoulder width of 12 ft [3.6 m] should be considered. On freeways with six or more lanes, the paved width of the right and left shoulder should be 10 ft [3.0 m]; where the DDHV for truck traffic exceeds 250 veh/h, a paved shoulder width of 12 ft [3.6 m] should be considered.

When necessary for freeways in mountainous terrain, the paved right shoulder may be reduced to 8 ft [2.4 m]. On four- or six-lane freeways, the paved left shoulder width may be reduced to 4 ft [1.2 m]. On freeways with eight or more lanes in mountainous terrain, the paved left and right shoulders should be at least 8 ft [2.4 m].

Guidance for ramp shoulder widths is provided in Section 10.9.6. Ramp shoulder widths are usually provided adjacent to acceleration and deceleration lanes with transitions to the freeway shoulder width at the taper ends. To facilitate drainage, shoulder cross slope should range between 2 and 6 percent and can be at least 1 percent greater than the pavement cross slope on tangent sections.

8.2.5 Curbs

Caution should be exercised in the use of curbs on freeways; where curbs are provided, they should not be closer to the traveled way than the outer edge of shoulder and should be easily traversable. An example of where shoulder curbs may be used on freeways is at locations where curbs are provided to control drainage and reduce erosion. For more information, refer to the discussion on curb types and their placement in Section 4.7 and the AASHTO *Roadside Design Guide* (*4*).

8.2.6 Superelevation

Maximum superelevation rates of 6 to 12 percent are applicable to horizontal curves on freeways. However, where snow and ice conditions are prevalent, a maximum rate of 6 to 8 percent should be considered. In these climates and where congestion or other factors result in recurrent slow-moving traffic, it is common practice to limit the superelevation rate to 6 percent. This may also be considered on viaducts where freezing and thawing conditions are likely, as bridge decks generally freeze more rapidly than other roadway sections. Where freeways are intermittently elevated on viaducts, a uniform maximum superelevation rate should be used throughout for design consistency.

The maximum cross-slope break between the traveled way and the shoulder should be limited to 8 percent to reduce the risk of truck rollover (*9*).

8.2.7 Grades

Maximum grades for freeways are presented in Table 8-1 for combinations of design speed and terrain type. Grades on freeways in urban areas should be comparable to those on freeways in rural areas of the same design speed. Steeper grades are permitted in urban areas, but the closer spacing of interchanges, the need for frequent speed changes, and the detrimental effect of steep grades on traffic flow make it desirable to use gentle grades wherever practical. On sustained upgrades, the need for climbing lanes should be investigated, as discussed in Section 3.4.3.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

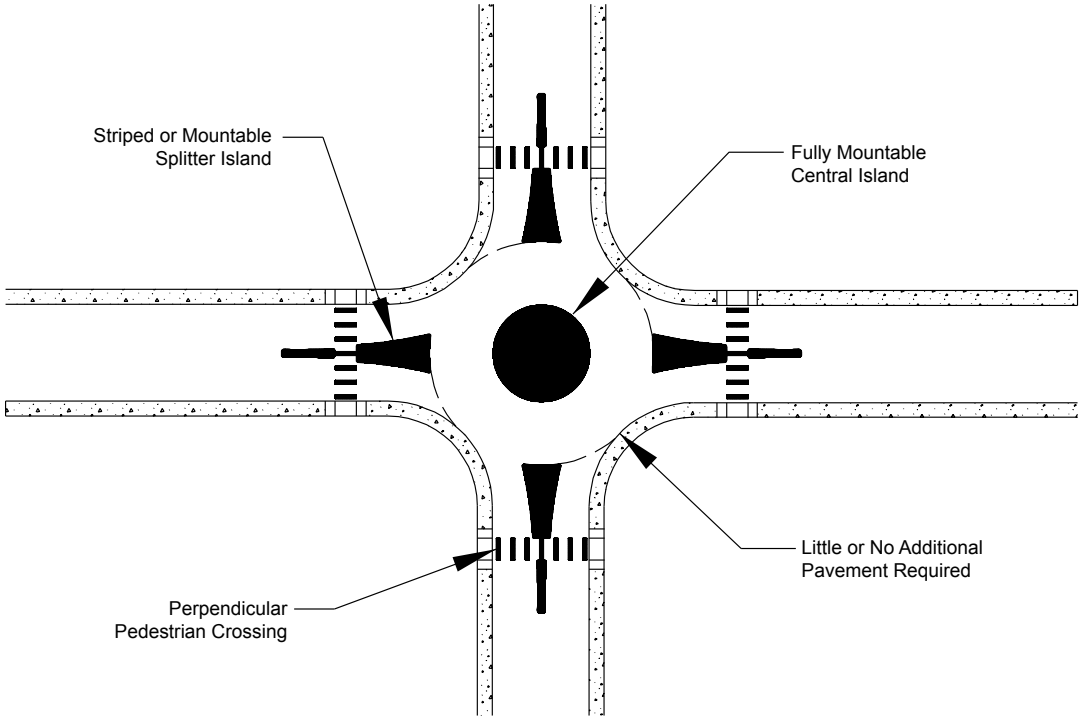


Figure 9-12. Typical Mini-Roundabout

9.3.4.2 Single-Lane Roundabouts

Single-lane roundabouts are characterized as having a single entry lane at all legs and one circulatory lane. Figure 9-13 provides an example of a typical single-lane roundabout in an urban area. They are distinguished from mini-roundabouts by their larger inscribed circle diameters and non-mountable central islands. Their design allows slightly higher speeds at the entry, on the circulatory roadway, and at the exit. The geometric design includes raised splitter islands, a non-mountable central island, and typically a truck apron. The size of the roundabout is largely influenced by the choice of design vehicle.

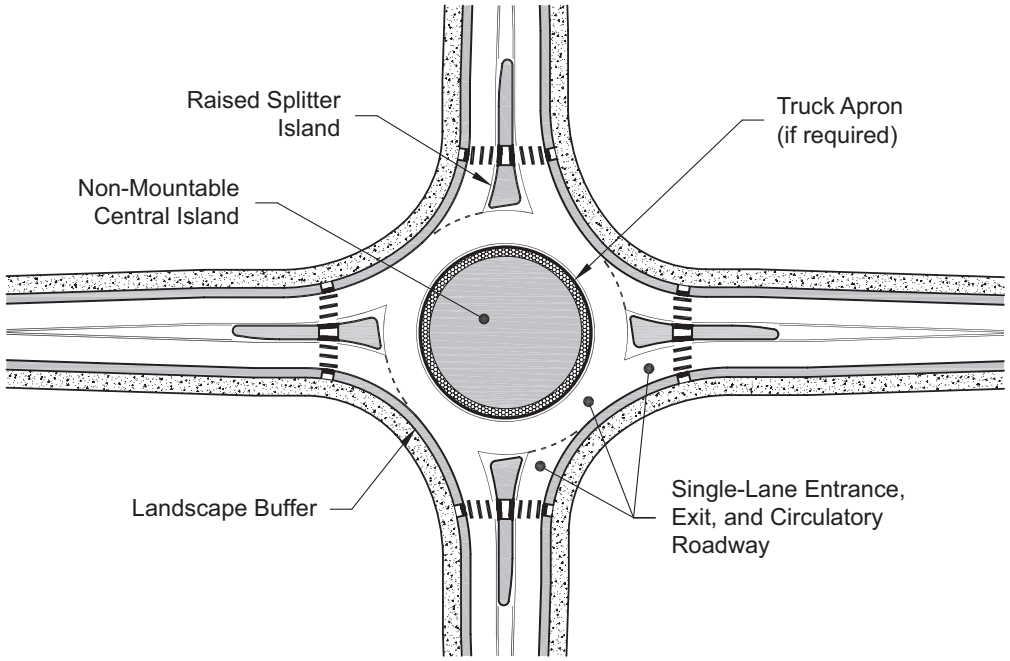


Figure 9-13. Typical Single-Lane Roundabout

9.3.4.3 Multilane Roundabouts

Multilane roundabouts include all roundabouts that have at least one entry with two or more lanes. In some cases, the roundabout may have a different number of lanes on one or more approaches. For example, a roundabout with both two-lane entries and single-lane entries would still be considered a multilane roundabout. They also include roundabouts with entries on one or more approaches that flare from one to two or more lanes. These need wider circulatory roadways to accommodate more than one vehicle travelling side-by-side. Figure 9-14 provides an example of a typical multilane roundabout. The speeds at the entry, on the circulatory roadway, and at the exit are similar to or may be slightly higher than those for the single-lane roundabouts. As with single-lane roundabouts, it is important that the vehicular speeds be consistent throughout the roundabout. The geometric design will include raised splitter islands, a truck apron, a non-mountable central island, and appropriate horizontal deflection.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

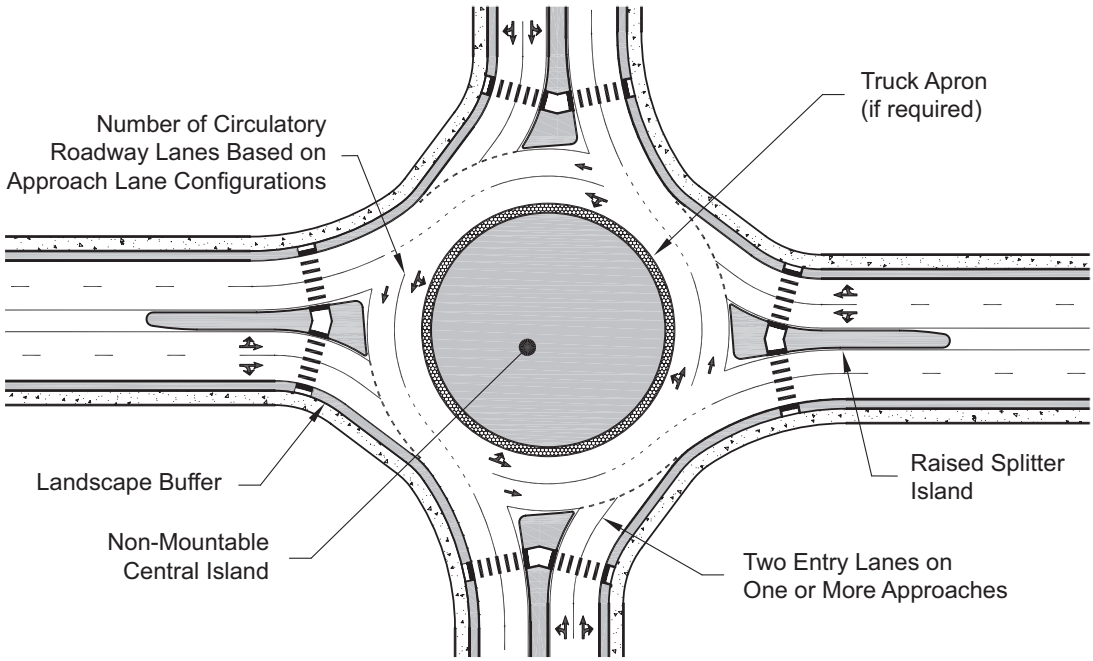


Figure 9-14. Typical Multilane Roundabout

9.4 ALIGNMENT AND PROFILE

9.4.1 General Considerations

Intersections are points of conflict between motor vehicles, pedestrians, and bicycles. The alignment and grade of the intersecting roads should permit users to easily recognize the intersection and vehicles using it and readily perform the maneuvers needed to pass through the intersection with minimum interference. To these ends, the alignment should be as straight and the gradients as flat as practical. The sight distance should be equal to or greater than the minimum values for specific intersection conditions, as discussed in Section 9.5 on "Intersection Sight Distance."

Site conditions generally establish definite alignment and grade constraints on the intersecting roads. It may be practical to modify the alignment and grades, however, in order to improve traffic operations.

9.4.2 Alignment

To reduce costs and crash frequencies, intersecting roads should generally meet at, or nearly at, right angles, unless roundabouts are utilized. Roads intersecting at acute angles need extensive

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

turning roadway areas and tend to limit visibility. Acute-angle intersections also increase the exposure time for the vehicles crossing the main traffic flow. The practice of realigning roads intersecting at acute angles in the manner shown in Figure 9-15A and 9-15B has proved to be beneficial. The greatest benefit is obtained when the curves used to realign the roads allow operating speeds nearly equivalent to the major-roadway approach speeds.

The practice of constructing short-radius horizontal curves on side-road approaches to achieve right-angle intersections should be avoided whenever practical. The intersection and traffic control devices at the intersection may be located outside the driver's line of sight, resulting in the need to install advanced signing. Sharp curves may also result in increased lane encroachments.

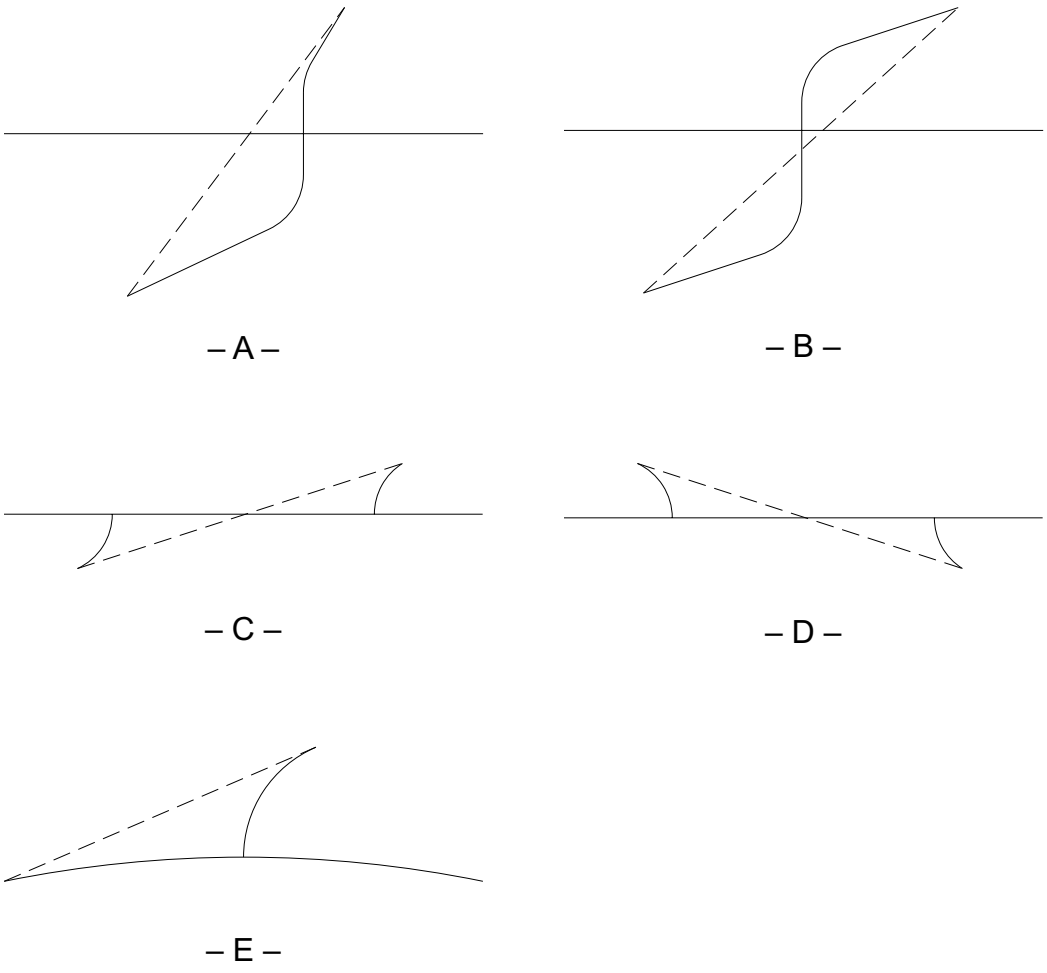


Figure 9-15. Realignment Variations at Intersections

Another method of realigning a road that originally intersected another road at an acute angle is to make an offset intersection, as shown in Figures 9-15C and 9-15D. A single curve is introduced on each crossroad leg to create two T-intersections such that crossing vehicles turn onto

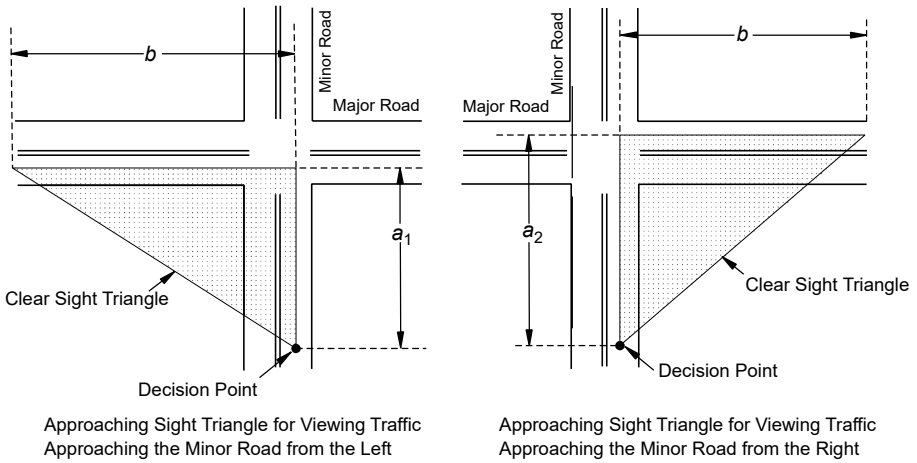


Figure 9-16—Approach Sight Triangles at Intersections

The vertex of the sight triangle on a minor-road approach (or an uncontrolled approach) represents the decision point for the minor-road driver (see Figure 9-16). This decision point is the location at which the minor-road driver should begin to brake to a stop if another vehicle is present on an intersecting approach. The distance from the major road, along the minor road, is illustrated by the distance a_1 to the left and a_2 to the right as shown in Figure 9-16. Distance a_2 is equal to distance a_1 plus the width of the lane(s) departing from the intersection on the major road to the right. Distance a_2 should also include the width of any median present on the major road unless the median is wide enough to permit a vehicle to stop before entering or crossing the roadway beyond the median.

The geometry of a clear sight triangle is such that when the driver of a vehicle without the right-of-way sees a vehicle that has the right of way on an intersecting approach, the driver of that potentially conflicting vehicle can also see the first vehicle. Distance b illustrates the length of this leg of the sight triangle. Thus, the provision of a clear sight triangle for vehicles without the right-of-way also permits the drivers of vehicles with the right-of-way to slow, stop, or avoid other vehicles, if needed.

Although desirable at higher volume intersections, approach sight triangles like those shown in Figure 9-16 are not needed for intersection approaches controlled by stop signs or traffic signals. In that case, the need for approaching vehicles to stop at the intersection is determined by the traffic control devices and not by the presence or absence of vehicles on the intersecting approaches.

9.5.2.2 Departure Sight Triangles

A second type of clear sight triangle provides sight distance sufficient for a stopped driver on a minor-road approach to depart from the intersection and enter or cross the major road. Figure 9-17 shows typical departure sight triangles to the left and to the right of the location of a stopped vehicle on the minor road. Departure sight triangles should be provided in each quadrant of each intersection approach controlled by stop or yield signs from which stopped vehicles may enter or cross a major road on which traffic is not required to stop. Departure sight triangles should also be provided for some signalized intersection approaches (see Section 9.5.3.4). Distance a_2 in Figure 9-17 is equal to distance a_1 plus the width of the lane(s) departing from the intersection on the major road to the right. Distance a_2 should also include the width of any median present on the major road unless the median is wide enough to permit a vehicle to stop before entering or crossing the roadway beyond the median. The appropriate measurement of distances a_1 and a_2 for departure sight triangles depends on the placement of any marked stop line that may be present and, thus, may vary with site-specific conditions.

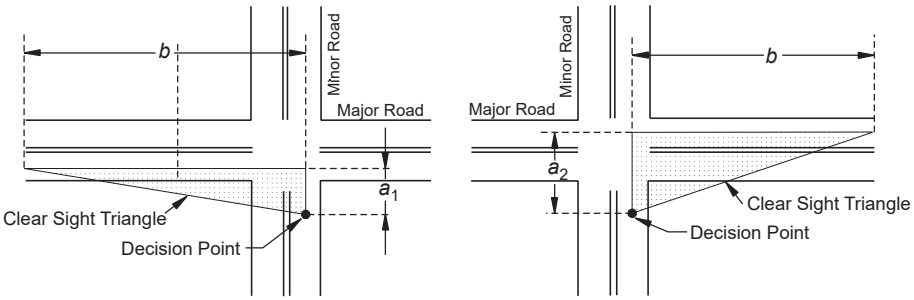


Figure 9-17. Departure Sight Triangles for Intersections

The recommended dimensions of the clear sight triangle for desirable traffic operations where stopped vehicles enter or cross a major road are based on assumptions derived from field observations of driver gap-acceptance behavior (*21*). The provision of clear sight triangles like those shown in Figure 9-17 also allows the drivers of vehicles on the major road to see any vehicles stopped on the minor-road approach and to be prepared to slow or stop, if needed.

9.5.2.3 Identification of Sight Obstructions within Sight Triangles

The profiles of the intersecting roadways should be designed to provide the recommended sight distances for drivers on the intersection approaches. Within a sight triangle, any object at a height above the elevation of the adjacent roadways that would obstruct the driver's view should

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

U.S. Customary	Metric
$c = \frac{V_o e^{-V_o t_c/3600}}{1 - e^{-V_o t_f/3600}}$ (9-3)	$c = \frac{V_o e^{-V_o t_c/3600}}{1 - e^{-V_o t_f/3600}}$ (9-3)
where:	where:
c = left-turn capacity, veh/h	c = left-turn capacity, veh/h
V_o = major-road volume conflicting with the minor movement, assumed to be equal to one-half of the two-way major-road volume, veh/h	V_o = major-road volume conflicting with the minor movement, assumed to be equal to one-half of the two-way major-road volume, veh/h
t_c = critical gap, s	t_c = critical gap, s
t_f = follow-up gap, s	t_f = follow-up gap, s

U.S. Customary	Metric
$SL = \left\{ \frac{\ln\left[P(n > N)\right]}{\ln\left[\frac{v}{c}\right]} - 1 \right\} \times VL$ (9-4)	$SL = \left\{ \frac{\ln\left[P(n > N)\right]}{\ln\left[\frac{v}{c}\right]} - 1 \right\} \times VL$ (9-4)
where:	where:
SL = storage length, ft	SL = storage length, m
$P(n>N)$ = probability of turn-lane overflow	$P(n>N)$ = probability of turn-lane overflow
v = left-turn vehicle volume, veh/h	v = left-turn vehicle volume, veh/h
c = left-turn capacity, veh/h	c = left-turn capacity, veh/h
VL = average length per vehicle, ft	VL = average length per vehicle, m

In applying these equations, $P(n>N)$, the probability that the number of vehicles stored will exceed the available length of the left-turn lane, is typically set equal to 0.005, equivalent to an assumption that the available storage length will accommodate the left-turning vehicle queue 99.5 percent of the time. The critical gap (t_c) is typically set equal to the 50th percentile value observed in field studies, 5.0 s, or the 85th percentile value observed in field studies, 6.25 s (*16*). The 85th percentile is suggested for design. The follow-up gap (t_f) is typically 2.5 s and the average storage length per vehicle is 25 ft [7.6 m].

Equations 9-3 and 9-4 show that the appropriate storage length is dependent on both the volume of turning traffic using the deceleration lane and the volume of opposing traffic. If volume data are not available, the minimum storage length should be at least 50 ft [16 m] to

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

accommodate two cars on urban and suburban streets with speeds less than 40 mph [70 km/h]. A minimum storage length of 100 ft [30 m] is recommended for high-speed and rural locations. Some cities use 250-ft [80-m] storage lanes for left-turn lanes approaching arterial streets and 150-ft [50 m] storage lanes for left-turn lanes approaching collector streets and most local streets, with a minimum length of 100 ft [30 m] at local streets and minor driveways.

Tables 9-21 and 9-22 provide computed values of storage length determined with Equations 9-3 and 9-4 and the typical assumptions presented above. If the percentage of trucks and buses is known, the minimum queue storage values from Tables 9-21 or 9-22 can be adjusted by multiplying by the values in Table 9-23. Traffic signal design fundamentals are discussed further in the MUTCD (*9*).

Table 9-21. Calculated Storage Lengths to Accommodate the 50th Percentile Critical Gap (*16*)

Left-Turn Volume (veh/h)	U.S. Customary					Metric				
	Storage Length (ft)					Storage Length (m)				
	Opposing Volume (veh/h)					Opposing Volume (veh/h)				
	200	400	600	800	1000	200	400	600	800	1000
40	50	50	50	50	50	16	16	16	16	16
60	50	50	50	50	50	16	16	16	16	16
80	50	50	50	50	50	16	16	16	16	16
100	50	50	50	50	75	16	16	16	16	23
120	50	50	50	75	75	16	16	16	23	23
140	50	50	50	75	75	16	16	16	23	23
160	50	50	75	75	100	16	16	23	23	31
180	50	50	75	75	100	16	16	23	23	31
200	50	75	75	100	125	16	23	23	31	39
220	50	75	75	100	125	16	23	23	31	39
240	75	75	100	125	150	23	23	31	39	46
260	75	75	100	125	175	23	23	31	39	54
280	75	75	100	125	175	23	23	31	39	54
300	75	100	125	150	200	23	31	39	46	61

Notes:

1. Storage lengths calculated from Equations 9-3 and 9-4 with a 0.005 probability of overflow.
2. Critical gap = 5.0 s; follow-up gap = 2.2 s.
3. Average storage length per vehicle is 25 ft [7.6 m]. Table 9-23 provides other suggested values for vehicle spacing based on percent trucks.

important concern. Additionally, many of the design techniques are substantially different for single-lane roundabouts than for roundabouts with two or more lanes.

9.10.1 Geometric Elements of Roundabouts

Figure 9-61 provides an overview of the basic geometric features and dimensions of a roundabout. These basic geometric elements are defined as follows:

Central island — The central island is the raised area in the center of a roundabout around which traffic circulates. The central island does not necessarily need to be circular in shape.

Splitter island — A splitter island is a raised or painted area on an approach used to separate entering from existing traffic, deflect and slow entering traffic, and allow pedestrians to cross the roadway in two stages.

Circulatory roadway — The circulatory roadway is the curved path used by vehicles to travel in a counterclockwise fashion around the central island.

Apron — If needed on smaller roundabouts to accommodate the wheel tracking of large vehicles, an apron is the mountable portion of the central island adjacent to the circulatory roadway.

Yield line at entrance to circulating roadway — The yield line marks the point of entry into the circulatory roadway. In most countries this line has the legal meaning of requiring entering motorists to yield the right of way; however, in the United States it is technically only an extension of the circulatory roadway edge line. Entering vehicles must yield to any circulating traffic coming from the left before crossing this line into the circulatory roadway.

Accessible pedestrian/ bicycle crossings — Accessible pedestrian crossings should be provided at all roundabouts. The crossing location is set back from the entrance line, and the splitter island is cut to allow pedestrians, wheelchairs, strollers, and bicycles to pass through.

Landscape strip — Landscape strips are provided at most roundabouts to separate vehicular and pedestrian traffic and to lead pedestrians to the designated crossing locations. Landscape strips can also significantly improve the aesthetics of the intersection.

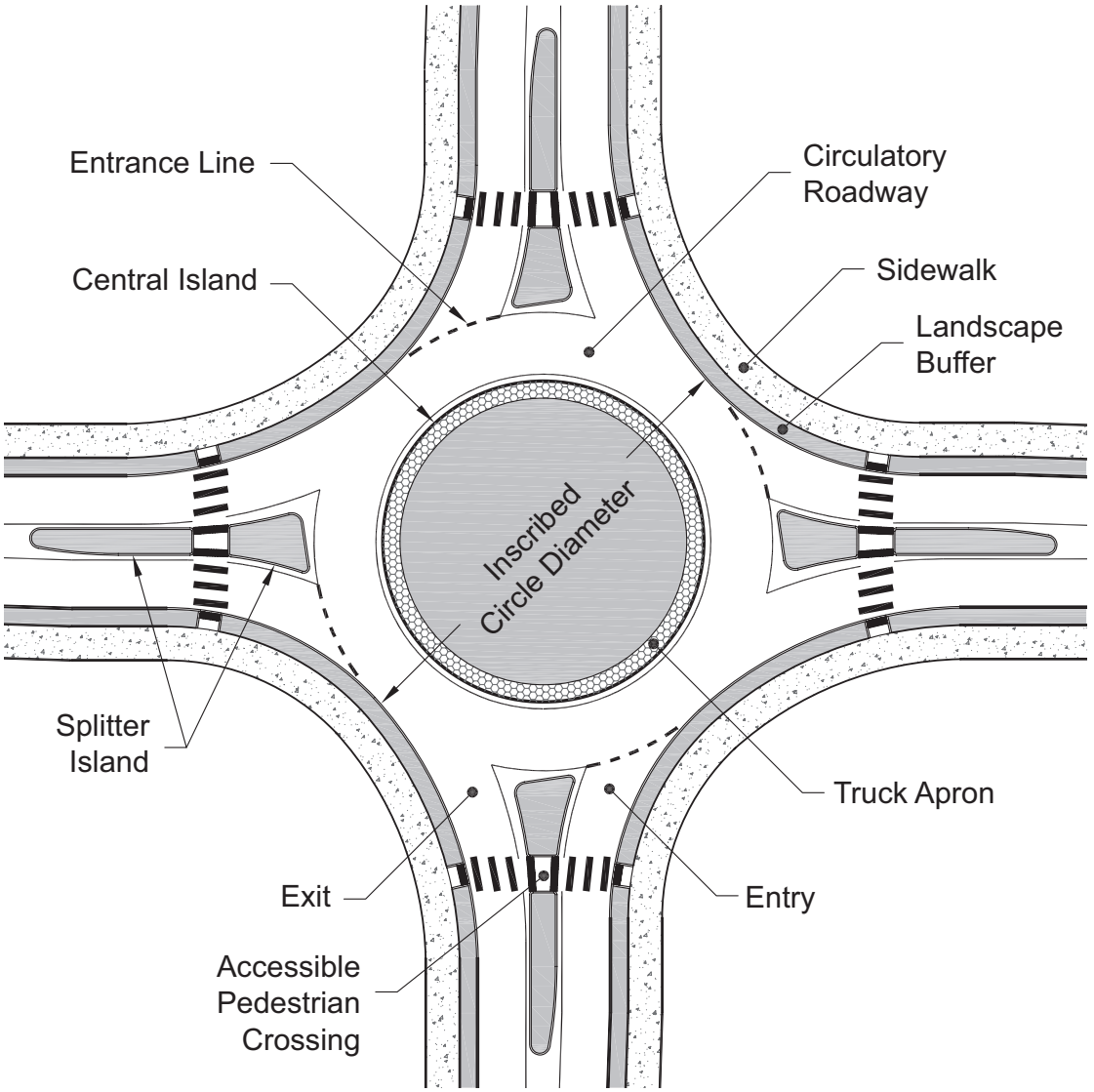


Figure 9-61. Basic Geometric Elements of a Roundabout

Key aspects of the geometric design of roundabouts are summarized below. Further details are presented in *Roundabouts: An Informational Guide* (*41*).

9.10.1.1 Size and Space Needs

The key indicator of the space needed for a roundabout intersection is the inscribed circle diameter. Table 9-3 in Section 9.3.4 provides ranges of inscribed circle diameters that may be used for accessing the range of potential effects. When large vehicles need to be accommodated, the inscribed circles would be near the high end of the range provided.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

exiting vehicles in the inside lane and left-turning vehicles that continue to circulate around in the outside lane.

The allowed movements assigned to each entering lane are key to the overall design. Basic pavement marking layouts should be considered integral to the preliminary design process so that lane continuity is being provided. In some cases, the geometry within the roundabout may be dictated by the number of lanes needed or the need to provide spiral transitions. Lane assignments should be clearly identified on all preliminary designs in an effort to retain the lane configuration information through the various design iterations.

In some cases, a roundabout designed to accommodate design year traffic volumes, typically projected 20 years from the present, can result in substantially more entering, exiting, and circulating lanes than needed in the earlier years of operation. Because the number of crashes may be higher with underutilized entering and circulating lanes, the designer may wish to consider a phased design solution. In this case, the first phase design would provide a single-lane entry to serve the near-term traffic volumes with the ability to easily expand the entries and circulatory roadway to accommodate future traffic volumes. To allow for expansion to the ultimate design at a later phase, the ultimate configuration of the roundabout needs to be considered in the initial phase.

Right-turn bypass lanes, also called slip lanes, can be implemented at roundabout intersections to increase the motor vehicle capacity. A bypass lane is a separate right-turn lane that lies adjacent to the roundabout and allows right-turning movements to bypass the roundabout. There are three configurations for the bypass lane: slip lane without an acceleration lane stop, slip lane without an acceleration lane yield, and slip lane with free-flow entry. In areas with bicycle and pedestrian activity, bypass lanes should be discouraged and should only be used where needed, since the entries and exits of bypass lanes can increase conflicts with pedestrians, bicyclists, and with merging on the downstream leg.

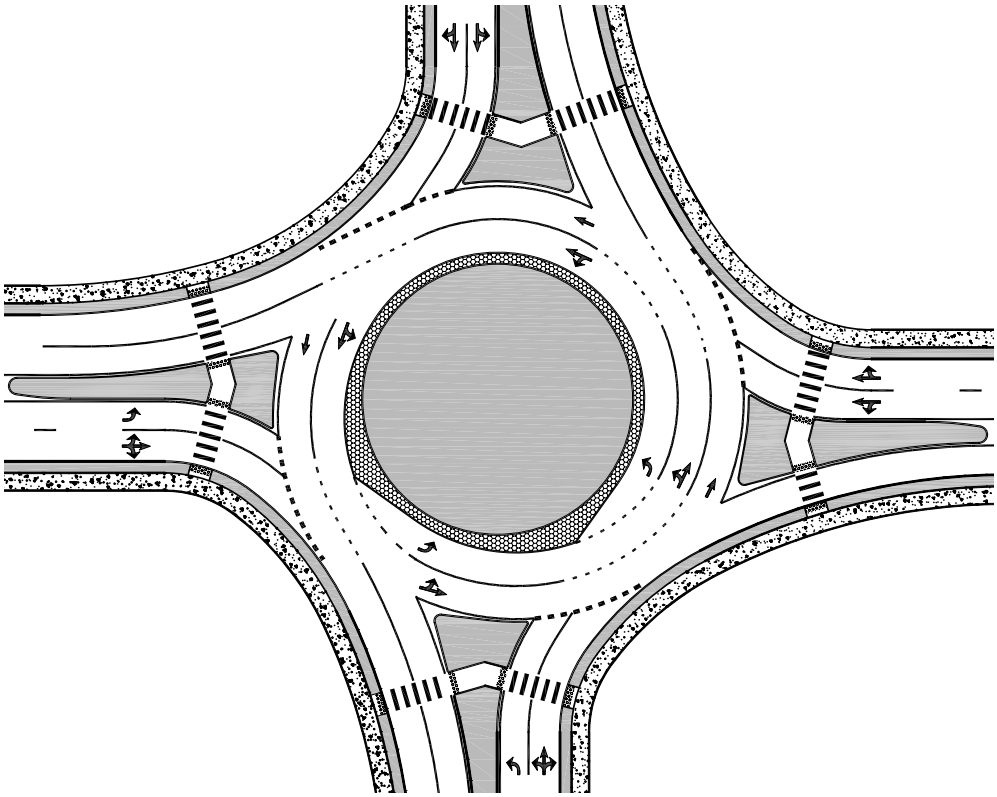


Figure 9-62. Roundabout Lane Configuration Example

9.10.2.3 Appropriate Natural Path Alignment

As two traffic streams approach the roundabout in adjacent lanes, vehicles will be guided by lane markings up to the entrance line. At the yield point, vehicles will continue along their natural trajectory into the circulating roadway. The speed and orientation of the vehicle at the entrance line determines its natural path. If the natural path of one lane interferes or overlaps with the natural path of the adjacent lane, the roundabout will not operate as efficiently. The geometry of the exits also affects the natural path that vehicles will travel. Overly small exit radii on multi-lane roundabouts may also result in overlapping vehicle paths on exit.

The fundamental principle related to natural vehicle path is that the entry design should align vehicles into the appropriate lane within the circulatory roadway. The design of exits should also provide appropriate alignment to allow drivers to intuitively maintain the appropriate lane. These alignment considerations often compete with the fastest path speed objectives; however, both of these fundamental principles should be achieved within the design process.

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

9.11.2 Traffic Control Devices

Traffic control devices are used to regulate, warn, and guide traffic and are a primary determinant in the efficient operation of intersections. It is essential that intersection design be accomplished simultaneously with the development of signal, signing, and pavement marking plans so that sufficient space is provided for proper installation of traffic control devices. Geometric design should not be considered complete nor should it be implemented until it has been determined that needed traffic devices will have the desired effect in controlling traffic.

Most of the intersection types illustrated and described in this chapter are adaptable to either signing control, signal control, or a combination of both. At intersections that do not need signal control, the normal roadway widths of the approach roadways are carried through the intersection with the possible addition of median lanes, auxiliary lanes, or pavement tapers. Where volumes are sufficient to indicate signal control, the number of lanes for through movements may also need to be increased. Where the volume approaches the uninterrupted flow capacity of the intersection leg, the number of lanes in each direction may have to be doubled at the intersection to accommodate the volume under stop-and-go control. Other geometric features that may be affected by signalization are length and width of storage areas, location and position of turning roadways, spacing of other subsidiary intersections, access connections, and the possible location and size of islands to accommodate signal posts or supports.

At high-volume intersections at grade, the design of the signals should be sophisticated enough to respond to the varying traffic demands, the objective being to keep the vehicles moving through the intersection. Factors affecting capacity and computation procedures for signalized intersections are covered in the HCM (*49*).

An intersection that needs traffic signal control is best designed by considering jointly the geometric design, capacity analysis, design hour volumes, and physical controls. Details on the design and location of most forms of traffic control signals, including the general warrants, are given in the MUTCD (*9*).

The number and arrangement of lanes, including the need for bicycle facilities, are crucial to successful operation of signalized intersections. The crossing distances for both vehicles and pedestrians should normally be kept as short as practical to reduce exposure to conflicting movements. Therefore, the first step in the development of intersection geometrics should be a complete analysis of current and future traffic demand, including pedestrian, bicycle, and transit users. The need to provide right- and left-turn lanes to minimize the interference of turning traffic with the movement of through traffic should be evaluated concurrently with the potential for obtaining any additional right-of-way needed. Along a roadway or street with a number of signalized intersections, the locations where turns will or will not be accommodated should also be examined to facilitate optimal traffic signal coordination.

9.11.3 Bicyclists

Where bicycle facilities enter an intersection, the design of the intersection should incorporate the bicycle facility. Intersection features compatible with bicycle facilities include: special sight distance considerations, wider roadways to accommodate on-street lanes, special lane markings to channelize and separate bicycles from right-turning vehicles, provisions for left-turn bicycle movements, or special traffic signal designs (such as bicycle detection at actuated signals or separate signal indications for bicyclists). Further guidance in providing for bicycles at intersections can be found in the AASHTO *Guide for the Development of Bicycle Facilities* (*3*) and the FHWA *Separated Bike Lane Planning and Design Guide* (*13*).

9.11.4 Pedestrians

Pedestrian facilities include sidewalks, crosswalks, traffic control features, and curb ramps for persons with disabilities that are also useful for people with baby strollers, wagons, carts, and luggage. Both marked and unmarked crosswalks should be considered in intersection design. Where sidewalks are present, the projected line of the sidewalk across an intersecting street constitutes a crosswalk, even where no crosswalk markings are present. When designing a project that involves curbs and adjacent sidewalks to accommodate pedestrian traffic, proper attention should be given to location and design of ramps and traffic control devices to accommodate the needs of persons with a variety of disabilities, such as mobility, vision, hearing, and cognitive disabilities. Related design criteria and illustrations are given in Section 4.17. Pedestrian facilities must be designed so that they are accessible to and usable by individuals with disabilities *52*, *53*). Further guidance in providing for pedestrians at intersections can be found in the AASHTO *Guide for the Planning, Design, and Operation of Pedestrian Facilities* (*1*) and *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way* (*51*).

9.11.5 Lighting

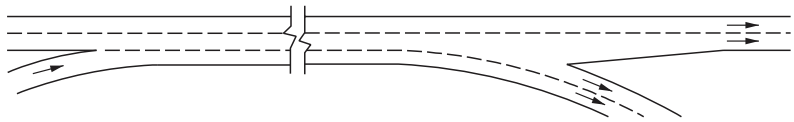
Lighting may reduce crashes at roadway and street intersections, as well as increase the efficiency of traffic operations. Statistics indicate that the nighttime crash rates are higher than that during daylight hours. This fact, to a large degree, may be attributed to impaired visibility. In urban and suburban areas where there are concentrations of pedestrians and roadside and intersectional interferences, fixed-source lighting tends to reduce crashes. Whether or not intersections in the rural context should be lighted depends on the planned geometrics and the turning volumes involved. Intersections that are not channelized are seldom lighted. However, for the benefit of nonlocal roadway users, lighting at intersections in the rural context is desirable to aid the driver in ascertaining sign messages during non-daylight periods.

Intersections with channelization, including roundabouts, should include lighting. Large channelized intersections especially need illumination because of the higher range of turning radii that are not within the lateral range of vehicular headlight beams. Vehicles approaching the

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

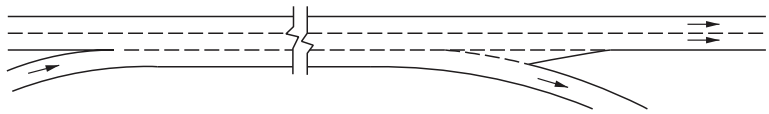
lence to the through lanes. When an auxiliary lane is carried through one or more interchanges, it may be eliminated by lane reduction beyond the influence of the last interchange, beginning approximately 1,500 to 2,500 ft [450 to 750 m] downstream of the last acceleration lane (see Figure 10-54D).

Where interchanges are widely spaced, it may not be practical or necessary to extend the auxiliary lane from one interchange to the next. In such cases, the auxiliary lane originating at a two-lane entrance should be carried along the freeway for an effective distance beyond the merging point, as shown in Figures 10-55A1 and 10-55A2. An auxiliary lane introduced for a two-lane exit should be carried along the freeway for an effective distance in advance of the exit and then extended onto the ramp, as shown in Figures 10-55B1 and 10-55B2. Figures 10-55A1 and 10-55B1 show parallel designs, whereas Figures 10-55A2 and 10-55B2 show tapered designs.



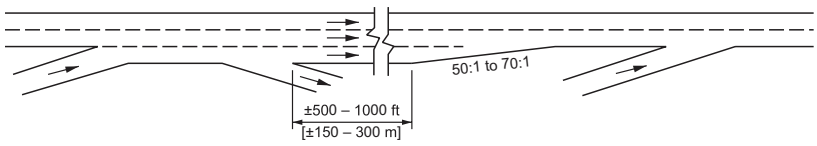
Auxiliary Lane Dropped on Exit Ramp

– A –



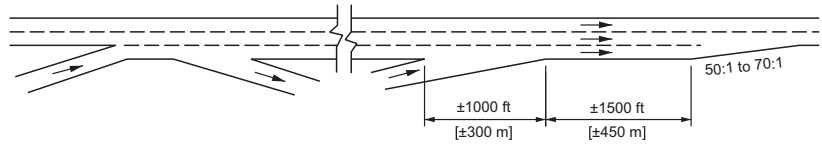
Auxiliary Lane between Cloverleaf Loops or Closely Spaced Interchanges Dropped on Single Exit Lane

– B –



Auxiliary Lane Dropped within an Interchange

– C –

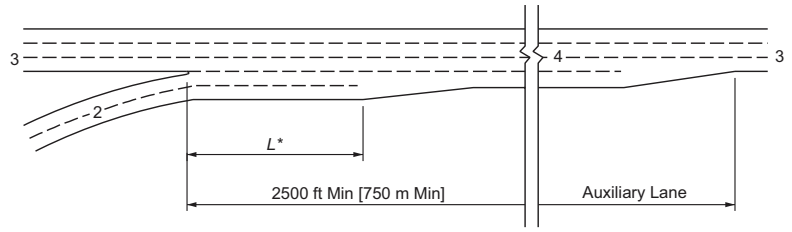


Auxiliary Lane Dropped beyond an Interchange

– D –

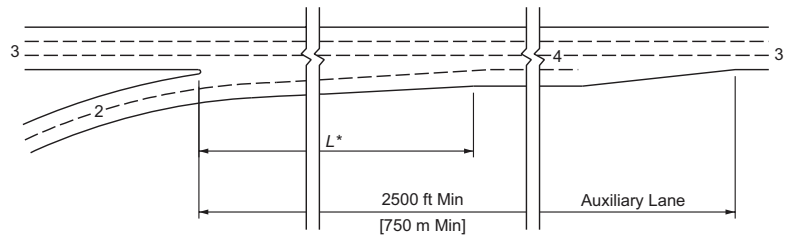
Figure 10-54. Alternative Methods of Reducing or Dropping Auxiliary Lanes

AUXILIARY LANE EXTENDED FOR EFFECTIVE DISTANCE BEYOND ENTRANCE



Parallel Design (Preferred)

– A1 –

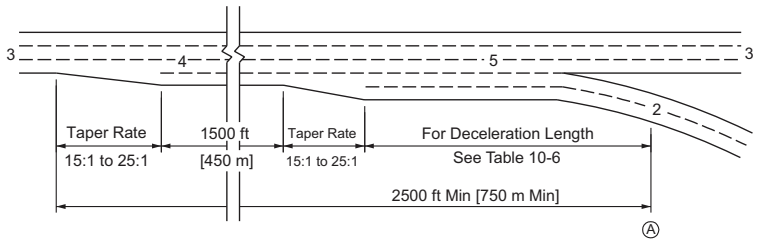


Tapered Design

– A2 –

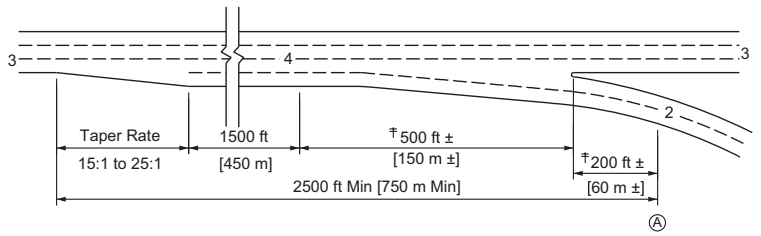
* Refer to Figure 10-76 for minimum length criteria.

AUXILIARY LANE INTRODUCED FOR EFFECTIVE DISTANCE IN ADVANCE OF EXIT



Parallel Design (Preferred)

– B1 –



Tapered Design

– B2 –

∓ Varies with angle of divergence
Ⓐ Point controlling speed on ramp

Figure 10-55. Coordination of Lane Balance and Basic Number of Lanes through Application of Auxiliary Lanes

© 2018 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.