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.
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:

\[ R_{pi} = \frac{V_e^2}{1.15 e_{max}} \]

In U.S. Customary Equation 3-11, changed multiplier in the denominator to “0.15”:

\[ R_{pi} = \frac{V_e^2}{0.15 e_{max}} \]

3-49 For \( e_{max} = 12\% \), \( e = 5.8\% \), and \( V = 45 \text{ mph} \), Table 3-12 shows a radius of 1,920 ft. In Table 3-12, for \( e_{max} = 12\% \), \( e = 5.8\% \), and \( V = 45 \text{ mph} \), 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.”

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.
U.S. Customary Equation 3-34 shows “0.1” just after the equal sign:

\[ Z = 0.1\left( \frac{V}{\sqrt{R}} \right) \]

In U.S. Customary Equation 3-34, deleted “0.1” just after the equal sign:

\[ Z = \left( \frac{V}{\sqrt{R}} \right) \]

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.

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.”

Metric Figure 3-15 is incorrectly positioned.

Rotated metric Figure 3-15 by 90 degrees.

Metric Figure 3-16 has a y axis label of “mph.”

Changed metric Figure 3-16’s y axis label to “km/h.”

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.”

In the last paragraph, the callout is “Figure 3-18.”

Changed the callout from “Figure 3-18” to “Figure 3-17.”

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.

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.”

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.

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.”

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.”
<table>
<thead>
<tr>
<th>Chapter</th>
<th>Page</th>
<th>Existing</th>
<th>Corrected</th>
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<tbody>
<tr>
<td>Chapter 6</td>
<td>6-25</td>
<td>In Section 6.4, reference entry 7, the title is incomplete.</td>
<td>In Section 6.4, reference entry 7, changed the title from “Drainage Manual” to “AASHTO Drainage Manual.”</td>
</tr>
<tr>
<td>Chapter 7</td>
<td>7-71</td>
<td>In Section 7.4, reference entry 11, the publication number has a zero where a hyphen should be.</td>
<td>In Section 7.4, reference entry 11, changed “FHWA-HEP017-024” to “FHWA-HEP-17-024.”</td>
</tr>
<tr>
<td>Chapter 8</td>
<td>8-4</td>
<td>Section 8.2.4 did not allow narrower shoulders in mountainous terrain as is allowed in the Interstate standard.</td>
<td>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].</td>
</tr>
<tr>
<td>Chapter 9</td>
<td>9-30</td>
<td>Figure 9-13, Typical Single-Lane Roundabout, is out of date.</td>
<td>Replaced Figure 9-13, Typical Single-Lane Roundabout.</td>
</tr>
<tr>
<td></td>
<td>9-31</td>
<td>Figure 9-14, Typical Multilane Roundabout, is out of date.</td>
<td>Replaced Figure 9-14, Typical Multilane Roundabout.</td>
</tr>
<tr>
<td></td>
<td>9-38</td>
<td>In Section 9.5.2.2, paragraph 1, sentence 3 is incomplete.</td>
<td>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.”</td>
</tr>
<tr>
<td></td>
<td>9-97</td>
<td>In both the U.S. Customary and metric versions of Equation 9-3, the variable “$t_c$” in the denominator is incorrect.</td>
<td>In both the U.S. Customary and metric versions of Equation 9-3, changed the variable “$t_c$” in the denominator to “$t_f$.”</td>
</tr>
<tr>
<td></td>
<td>9-144</td>
<td>Figure 9-61, Basic Geometric Elements of a Roundabout, is out of date.</td>
<td>Replaced Figure 9-61, Basic Geometric Elements of a Roundabout.</td>
</tr>
<tr>
<td></td>
<td>9-148</td>
<td>Figure 9-62, Roundabout Lane Configuration Example, is out of date.</td>
<td>Replaced Figure 9-62, Roundabout Lane Configuration Example.</td>
</tr>
<tr>
<td></td>
<td>9-156</td>
<td>In Section 9.11.4, the last reference callout is incorrect.</td>
<td>In Section 9.11.4, changed the last reference callout from “(50)” to “(51).”</td>
</tr>
<tr>
<td>Chapter 10</td>
<td>10-92</td>
<td>In Figure 10-55(B1), the outside edges of lane lines are missing.</td>
<td>In Figure 10-55(B1), replaced the missing outside edges of lane lines.</td>
</tr>
</tbody>
</table>
underpasses 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 proportionally 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.
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


10. AASHTO. AASHTOWare Safety Analyst software. Available at www.aashtoware.org and www.safetyanalyst.org


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

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


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

<table>
<thead>
<tr>
<th>U.S. Customary</th>
<th>Metric</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.01e + f = \frac{V^2}{15R}$</td>
<td>$0.01e + f = \frac{V^2}{127R}$</td>
</tr>
</tbody>
</table>

where:

- $V = V_D =$ design speed, mph
- $e = e_{\text{max}} =$ maximum superelevation, percent
- $f = f_{\text{max}} =$ maximum allowable side friction factor
- $R = R_{\text{min}} =$ minimum radius, ft

then:

$$R_{\text{min}} = \frac{V_D^2}{15(0.01e_{\text{max}} + f_{\text{max}})}$$

and where:

- $V = V_R =$ running speed, mph
- $R = R_{\text{PI}} =$ 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 $0.01e_{\text{max}}$ and $(0.01e + f)_R$)

then:

$$R_{\text{PI}} = \frac{V_R^2}{0.15e_{\text{max}}}$$

Because $(0.01e + f)_D - (0.01e + f)_R = h$, at point $R_{\text{PI}}$ the equations reduce to the following:

$$h_{\text{PI}} = \left(0.01e_{\text{max}}\right) \frac{V_D^2}{V_R^2} - 0.01e_{\text{max}}$$

where $h_{\text{PI}} =$ PI offset from the $1/R$ axis.

Also:

$$S_1 = \frac{h_{\text{PI}}(R_{\text{PI}})}{5729.58}$$
### Table 3-12. Minimum Radii for Design Superelevation Rates, Design Speeds, and $e_{\text{max}} = 12\%$

<table>
<thead>
<tr>
<th>$e$ (%)</th>
<th>$V_d = 15$ mph</th>
<th>$V_d = 20$ mph</th>
<th>$V_d = 25$ mph</th>
<th>$V_d = 30$ mph</th>
<th>$V_d = 35$ mph</th>
<th>$V_d = 40$ mph</th>
<th>$V_d = 45$ mph</th>
<th>$V_d = 50$ mph</th>
<th>$V_d = 55$ mph</th>
<th>$V_d = 60$ mph</th>
<th>$V_d = 65$ mph</th>
<th>$V_d = 70$ mph</th>
<th>$V_d = 75$ mph</th>
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<tr>
<td>R (ft)</td>
<td>950 1690 2460 3370 4390 5580 6910</td>
<td>9370 9990 11800 13200 14800 16400 18100</td>
<td>12250 13200 14800 16400 18100 20200 22100</td>
<td>24200 26200 28400 30800 33400 36200 39200</td>
<td>42400 45800 49400 53200 57200 61400 65800</td>
<td>69500 74000 78700 83600 88700 94000 99500</td>
<td>105100 111000 117200 123700 130500 137500 144800</td>
<td>152400 160300 168600 177300 186500 196200 206300</td>
<td>216800 228000 240600 253700 267300 281400 296000</td>
<td>311100 327200 344700 363600 383900 405500 428600</td>
<td>453200 480400 509400 540100 573500 609600 648600</td>
<td>690400 734900 782100 832900 887300 945400 1007900</td>
<td>1074900 1146400 1222500 1303200 1388500 1479500 1576100</td>
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</table>

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

<table>
<thead>
<tr>
<th>e (%)</th>
<th>$V_d = 15$ mph</th>
<th>$V_d = 20$ mph</th>
<th>$V_d = 25$ mph</th>
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</table>

| 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

| V_m (mph) | V = 20 | V = 25 | V = 30 | V = 35 | V = 40 | V = 45 | V = 50 | V = 55 | V = 60 | V = 65 | V = 70 | V = 75 | V = 80 | V = 85 |
|-----------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|
| e (%)     |        |        |        |        |        |        |        |        |        |        |        |        |        |        |
| 1.5       | 1      | 2      | 1      | 2      | 1      | 2      | 1      | 2      | 1      | 2      | 1      | 2      | 1      | 2      |
| 2.0       | 1      | 2      | 0.5    | 1      | 2      | 0.5    | 1      | 2      | 0.5    | 1      | 2      | 0.5    | 1      | 2      |
| 2.5       | 0.5    | 1      | 0.25   | 0.5    | 1      | 0.25   | 0.5    | 1      | 0.25   | 0.5    | 1      | 0.25   | 0.5    | 1      |
| 3.0       | 0.25   | 0.5    | 0.125  | 0.25   | 0.5    | 0.125  | 0.25   | 0.5    | 0.125  | 0.25   | 0.5    | 0.125  | 0.25   | 0.5    |
| 3.5       | 0.125  | 0.25   | 0.0625 | 0.125  | 0.25   | 0.0625 | 0.125  | 0.25   | 0.0625 | 0.125  | 0.25   | 0.0625 | 0.125  | 0.25   |
| 4.0       | 0.0625 | 0.125  | 0.03125| 0.0625 | 0.125  | 0.03125| 0.0625 | 0.125  | 0.03125| 0.0625 | 0.125  | 0.03125| 0.0625 | 0.125  |
| 4.5       | 0.03125| 0.0625 | 0.015625| 0.03125| 0.0625 | 0.015625| 0.03125| 0.0625 | 0.015625| 0.03125| 0.0625 | 0.015625| 0.03125| 0.0625 |
| 5.0       | 0.015625| 0.03125| 0.0078125| 0.015625| 0.03125| 0.0078125| 0.015625| 0.03125| 0.0078125| 0.015625| 0.03125| 0.0078125| 0.015625| 0.03125|
| 5.5       | 0.0078125| 0.015625| 0.00390625| 0.0078125| 0.015625| 0.00390625| 0.0078125| 0.015625| 0.00390625| 0.0078125| 0.015625| 0.00390625| 0.0078125| 0.015625|

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.)
### Table 3.6a. Superelevation Runoff $L_i$, (ft) for Horizontal Curves (Continued)

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<th>$v_i$ (mph)</th>
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Table 3-16b. Superelevation Runoff $L_r (m)$ for Horizontal Curves

| $V_d$ (km/h) | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 |
| 20           | 1.5| 7  | 10 | 7  | 10 | 7  | 11 | 8  | 12 | 9  | 13 | 9  | 14 | 11 | 16 | 11 | 16 | 11 | 16 | 11 | 16 |
| 30           | 2.0| 9  | 14 | 9  | 14 | 10 | 15 | 11 | 16 | 12 | 18 | 13 | 19 | 14 | 22 | 14 | 22 | 14 | 22 | 14 | 22 |
| 40           | 2.2| 10 | 15 | 10 | 15 | 11 | 16 | 12 | 18 | 13 | 19 | 14 | 21 | 15 | 23 | 16 | 24 | 16 | 24 | 16 | 24 |
| 50           | 2.4| 11 | 16 | 11 | 16 | 12 | 18 | 13 | 19 | 14 | 21 | 15 | 23 | 16 | 26 | 17 | 27 | 17 | 27 | 17 | 27 |
| 60           | 2.6| 12 | 18 | 12 | 18 | 13 | 19 | 14 | 21 | 15 | 23 | 16 | 25 | 17 | 28 | 18 | 29 | 18 | 29 | 18 | 29 |
| 70           | 2.8| 13 | 19 | 13 | 19 | 14 | 21 | 15 | 23 | 16 | 25 | 17 | 26 | 18 | 29 | 19 | 29 | 19 | 29 | 19 | 29 |
| 80           | 3.0| 14 | 20 | 14 | 20 | 15 | 22 | 16 | 24 | 17 | 26 | 18 | 29 | 19 | 29 | 20 | 30 | 20 | 30 | 20 | 30 |
| 90           | 3.2| 14 | 22 | 14 | 22 | 16 | 24 | 17 | 26 | 18 | 29 | 19 | 29 | 20 | 30 | 20 | 30 | 20 | 30 | 20 | 30 |
| 100          | 3.4| 15 | 23 | 15 | 23 | 17 | 25 | 18 | 28 | 20 | 30 | 21 | 32 | 23 | 34 | 26 | 39 | 26 | 39 | 26 | 39 |
| 110          | 3.6| 16 | 24 | 16 | 24 | 18 | 27 | 19 | 29 | 21 | 32 | 23 | 34 | 26 | 39 | 26 | 39 | 26 | 39 | 26 | 39 |
| 120          | 3.8| 17 | 26 | 17 | 26 | 19 | 28 | 21 | 31 | 22 | 33 | 24 | 36 | 27 | 41 | 27 | 41 | 27 | 41 | 27 | 41 |
| 130          | 4.0| 18 | 27 | 18 | 27 | 20 | 30 | 22 | 32 | 23 | 35 | 25 | 38 | 29 | 43 | 29 | 43 | 29 | 43 | 29 | 43 |
| 140          | 4.2| 19 | 28 | 19 | 28 | 21 | 31 | 23 | 34 | 25 | 37 | 27 | 40 | 29 | 43 | 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 |
|               | 4.6| 21 | 31 | 21 | 31 | 23 | 34 | 25 | 37 | 27 | 40 | 29 | 43 | 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 |
|               | 5.0| 23 | 34 | 23 | 34 | 25 | 37 | 27 | 40 | 29 | 44 | 32 | 47 | 36 | 54 | 36 | 54 | 36 | 54 | 36 | 54 |
|               | 5.2| 24 | 35 | 24 | 35 | 27 | 40 | 29 | 44 | 32 | 47 | 34 | 51 | 39 | 58 | 39 | 58 | 39 | 58 | 39 | 58 |
|               | 5.4| 25 | 36 | 25 | 36 | 27 | 40 | 29 | 44 | 32 | 47 | 34 | 51 | 39 | 58 | 39 | 58 | 39 | 58 | 39 | 58 |
|               | 5.6| 26 | 37 | 26 | 37 | 28 | 40 | 29 | 44 | 32 | 47 | 34 | 51 | 39 | 58 | 39 | 58 | 39 | 58 | 39 | 58 |
|               | 5.8| 27 | 38 | 27 | 38 | 29 | 42 | 30 | 45 | 33 | 49 | 35 | 53 | 40 | 60 | 40 | 60 | 40 | 60 | 40 | 60 |
|               | 6.0| 28 | 39 | 28 | 39 | 30 | 42 | 31 | 46 | 35 | 53 | 40 | 60 | 40 | 59 | 40 | 59 | 40 | 59 | 40 | 59 |
|               | 6.2| 29 | 40 | 29 | 40 | 31 | 46 | 33 | 50 | 36 | 54 | 39 | 59 | 45 | 67 | 45 | 67 | 45 | 67 | 45 | 67 |
|               | 6.4| 30 | 41 | 30 | 41 | 32 | 43 | 33 | 51 | 37 | 57 | 42 | 63 | 46 | 71 | 46 | 71 | 46 | 71 | 46 | 71 |
|               | 6.6| 31 | 42 | 31 | 42 | 33 | 53 | 36 | 56 | 39 | 64 | 42 | 67 | 46 | 74 | 46 | 74 | 46 | 74 | 46 | 74 |
|               | 6.8| 32 | 43 | 32 | 43 | 34 | 55 | 37 | 60 | 40 | 63 | 42 | 69 | 46 | 77 | 46 | 77 | 46 | 77 | 46 | 77 |
Table 3-16b. Superelevation Runoff $L_r$ (m) for Horizontal Curves (Continued)

| $V_d$ (km/h) | Lr (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) |
|-------------|-------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|
| 30          | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 40          | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 50          | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 60          | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 70          | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 80          | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 90          | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 100         | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 110         | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 120         | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 130         | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 140         | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |
| 150         | 1.01  | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      | 1.01      |

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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:

\[
L_t = \frac{\epsilon_{NC}}{\epsilon_d} L_r
\]

where:
- \(L_t\) = minimum length of tangent runout, ft
- \(\epsilon_{NC}\) = normal cross slope rate, percent
- \(\epsilon_d\) = design superelevation rate, percent
- \(L_r\) = minimum length of superelevation runoff, ft

\[
L_t = \frac{\epsilon_{NC}}{\epsilon_d} L_r
\]

where:
- \(L_t\) = minimum length of tangent runout, m
- \(\epsilon_{NC}\) = normal cross slope rate, percent
- \(\epsilon_d\) = design superelevation rate, percent
- \(L_r\) = minimum length of superelevation runoff, m

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.
Figure 3-8. Diagrammatic Profiles Showing Methods of Attaining Superelevation for a Curve to the Right (Continued)
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 particu-
larly 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 obvi-
ously 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 min-
imum 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 tem-
plate, 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 aff-
ords 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 vol-
umes. 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.
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, \( W_n \), equal to 20, 22, and 24 ft [6.0, 6.6, and 7.2 m], respectively.

The width of the front overhang (\( F_A \)) 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, \( F_A \) 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 \( F_A \) determined from:

\[
F_A = \sqrt{R^2 + A(2L + A)} - R
\]

where:

- \( F_A \) = width of front overhang, ft
- \( R \) = radius of curve or turning roadway (two-lane), ft
- \( A \) = front overhang of inner lane vehicle, ft
- \( L \) = wheelbase of single unit or tractor, ft

\[ F_A = \sqrt{R^2 + A(2L + A)} - R \]

where:

- \( F_A \) = width of front overhang, m
- \( R \) = radius of curve or turning roadway (two-lane), m
- \( A \) = front overhang of inner lane vehicle, m
- \( L \) = wheelbase of single unit or tractor, m

(3-33)

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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 \( FB = 0.5 \text{ ft} [0.15 \text{ m}] \). 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:

\[
Z = \left( \frac{V}{\sqrt{R}} \right)
\]

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.
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)

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<th>Design Speed (mph)</th>
<th>Traveled way width = 22 ft</th>
<th>Design Speed (mph)</th>
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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.
Table 3-24b. Calculated and Design Values For Traveled Way Widening on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)

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<th>Radius of Curve (m)</th>
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<tr>
<td>70</td>
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<td>0.3</td>
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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 [50 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.
Figure 3-15. Speed–Distance Curves for a Typical Heavy Truck of 140 lb/hp [85 kg/kW] for Deceleration on Upgrades
Figure 3-16. Speed–Distance Curves for Acceleration of a Typical Heavy Truck of 140 lb/hp [85 kg/kW] on Upgrades and Downgrades
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.
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-
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.

![Diagram of alignment and profile relationships in roadway design](Figure 3-40. Alignment and Profile Relationships in Roadway Design (43))
This combination presents a poor appearance – the horizontal curve looks like a sharp angle.

When horizontal and vertical curves oppose, a very satisfactory appearance results.

The classic case of coordination between horizontal and vertical alignment in which the vertices of horizontal and vertical curves coincide, creating a rich effect of three-dimensional S-curves composed of convex and concave helices.

A legitimate case of coordination: one phase is skipped in the horizontal plane, but vertices still coincide. The long tangent in plan is softened by vertical curvature.

The upper line is an example of poor design because the alignment consists of a long tangent with short curves, whereas the balance between the curves and tangents in the lower alignment is the preferred design.

When horizontal and vertical curves coincide, a very satisfactory appearance results.

Very long flat curves, even where not required by a design speed and regardless of profile, also have a pleasing appearance when the central angle is very small.

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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.


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


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


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

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


6.4 REFERENCES


11. FHWA. Traffic Analysis Tools website. Available at

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


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


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.
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.
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.
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
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](image)

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 $a_1$ 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.

![Departure Sight Triangles (Stop-Controlled)](image)

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 (27). 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
Intersections

**U.S. Customary**

\[
c = \frac{V_o e^{-V_o t_c / 3600}}{1 - e^{-V_o t_f / 3600}}
\]

where:
- \(c\) = 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
- \(t_c\) = critical gap, s
- \(t_f\) = follow-up gap, s

**Metric**

\[
c = \frac{V_o e^{-V_o t_c / 3600}}{1 - e^{-V_o t_f / 3600}}
\]

where:
- \(c\) = 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
- \(t_c\) = critical gap, s
- \(t_f\) = follow-up gap, s

**U.S. Customary**

\[
SL = \left( \frac{\ln \left[ P(n > N) \right]}{\ln \left[ \frac{v}{c} \right]} - 1 \right) \times VL
\]

where:
- \(SL\) = storage length, ft
- \(P(n > N)\) = probability of turn-lane overflow
- \(v\) = left-turn vehicle volume, veh/h
- \(c\) = left-turn capacity, veh/h
- \(VL\) = average length per vehicle, ft

**Metric**

\[
SL = \left( \frac{\ln \left[ P(n > N) \right]}{\ln \left[ \frac{v}{c} \right]} - 1 \right) \times VL
\]

where:
- \(SL\) = storage length, m
- \(P(n > N)\) = probability of turn-lane overflow
- \(v\) = left-turn vehicle volume, veh/h
- \(c\) = left-turn capacity, veh/h
- \(VL\) = 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
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)

<table>
<thead>
<tr>
<th>Left-Turn Volume (veh/h)</th>
<th>U.S. Customary</th>
<th>Metric</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Storage Length (ft)</td>
<td>Storage Length (m)</td>
</tr>
<tr>
<td></td>
<td>Opposing Volume (veh/h)</td>
<td>Opposing Volume (veh/h)</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>400</td>
</tr>
<tr>
<td>40</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>60</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>80</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>100</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>120</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>140</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>160</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>180</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>200</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>220</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>240</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>260</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>280</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>300</td>
<td>75</td>
<td>100</td>
</tr>
</tbody>
</table>

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:

<table>
<thead>
<tr>
<th><strong>Central island</strong></th>
<th>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.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Splitter island</strong></td>
<td>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.</td>
</tr>
<tr>
<td><strong>Circulatory roadway</strong></td>
<td>The circulatory roadway is the curved path used by vehicles to travel in a counterclockwise fashion around the central island.</td>
</tr>
<tr>
<td><strong>Apron</strong></td>
<td>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.</td>
</tr>
<tr>
<td><strong>Yield line at entrance to circulating roadway</strong></td>
<td>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.</td>
</tr>
<tr>
<td><strong>Accessible pedestrian/bicycle crossings</strong></td>
<td>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.</td>
</tr>
<tr>
<td><strong>Landscape strip</strong></td>
<td>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.</td>
</tr>
</tbody>
</table>
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.
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.
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.
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
ence 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](image1)

![Auxiliary Lane between Cloverleaf Loops or Closely Spaced Interchanges Dropped on Single Exit Lane](image2)

![Auxiliary Lane Dropped within an Interchange](image3)

![Auxiliary Lane Dropped beyond an Interchange](image4)

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

– A2 –

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

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