3 Elements of Design

3.1 INTRODUCTION

The alignment of a highway or street produces a great impact on the environment, the fabric of the community, and the highway user. The alignment consists of a variety of design elements that combine to create a facility that serves traffic safely and efficiently, consistent with the facility's intended function. Each alignment element should complement others to achieve a consistent, safe, and efficient design.

The design of highways and streets within particular functional classes is treated separately in later chapters. Common to all classes of highways and streets are several principal elements of design. These include sight distance, superelevation, traveled way widening, grades, horizontal and vertical alignments, and other elements of geometric design. These alignment elements are discussed in this chapter, and, as appropriate, in the later chapters pertaining to specific highway functional classes.

3.2 SIGHT DISTANCE

3.2.1 General Considerations

A driver's ability to see ahead is needed for safe and efficient operation of a vehicle on a highway. For example, on a railroad, trains are confined to a fixed path, yet a block signal system and trained operators are needed for safe operation. In contrast, the path and speed of motor vehicles on highways and streets are subject to the control of drivers whose ability, training, and experience are quite varied. The designer should provide sight distance of sufficient length that drivers can control the operation of their vehicles to avoid striking an unexpected object in the traveled way. Certain two-lane highways should also have sufficient sight distance to enable drivers to use the opposing traffic lane for passing other vehicles without interfering with oncoming vehicles. Two-lane rural highways should generally provide such passing sight distance at frequent intervals and for substantial portions of their length. On the other hand, it is normally of little practical value to provide passing sight distance on two-lane urban streets or arterials. The proportion of a highway's length with sufficient sight distance to pass another vehicle and interval between passing opportunities should be compatible with the intended function of the highway and the desired level of service. Design criteria and guidance applicable to specific functional classifications of highways and streets are presented in Chapters 5 through 8.

Four aspects of sight distance are discussed below: (1) the sight distances needed for stopping, which are applicable on all highways; (2) the sight distances needed for the passing of overtaken vehicles, applicable only on two-lane highways; (3) the sight distances needed for decisions at complex locations; and (4) the criteria for measuring these sight distances for use in design. The design of alignment and profile to provide sight distances and to satisfy the applicable design criteria are described later in this chapter. The special conditions related to sight distances at intersections are discussed in Section 9.5.

3.2.2 Stopping Sight Distance

Sight distance is the length of the roadway ahead that is visible to the driver. The available sight distance on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although greater lengths of visible roadway are desirable, the sight distance at every point along a roadway should be at least that needed for a below-average driver or vehicle to stop.

Stopping sight distance is the sum of two distances: (1) the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied, and (2) the distance needed to stop the vehicle from the instant brake application begins. These are referred to as brake reaction distance and braking distance, respectively.

Brake Reaction Time

Brake reaction time is the interval from the instant that the driver recognizes the existence of an obstacle on the roadway ahead that necessitates braking until the instant that the driver actually applies the brakes. Under certain conditions, such as emergency situations denoted by flares or flashing lights, drivers accomplish these tasks almost instantly. Under most other conditions, the driver needs not only to see the object but also to recognize it as a stationary or slowly moving object against the background of the roadway and other objects, such as walls, fences, trees, poles, or bridges. Such determinations take time, and the amount of time needed varies considerably with the distance to the object, the visual acuity of the driver, the natural rapidity with which the driver reacts, the atmospheric visibility, the type and the condition of the roadway, and nature of the obstacle. Vehicle speed and roadway environment probably also influence reaction time. Normally, a driver traveling at or near the design speed is more alert than one traveling at a lesser speed. A driver on an urban street confronted by innumerable potential conflicts with parked vehicles, driveways, and cross streets is also likely to be more alert than the same driver on a limited-access facility where such conditions should be almost nonexistent.

The study of reaction times by Johansson and Rumar (*39*) referred to in Section 2.2.6 was based on data from 321 drivers who expected to apply their brakes. The median reaction-time value for these drivers was 0.66 s, with 10 percent using 1.5 s or longer. These findings correlate with those of earlier studies in which alerted drivers were also evaluated. Another study (*44*) found 0.64 s as the average reaction time, while 5 percent of the drivers needed over 1 s. In a third study (*48*), the values of brake reaction time ranged from 0.4 to 1.7 s. In the Johansson and Rumar study (*39*), when the event that prompted application of the brakes was unexpected, the drivers' response times were found to increase by approximately 1 s or more; some reaction times were greater than 1.5 s. This increase in reaction time substantiated earlier laboratory and road tests in which the conclusion was drawn that a driver who needed 0.2 to 0.3 s of reaction time under alerted conditions would need 1.5 s of reaction time under normal conditions.

Minimum brake reaction times for drivers could thus be at least 1.64 s, 0.64 s for alerted drivers plus 1 s for the unexpected event. Because the studies discussed above used simple prearranged signals, they represent the least complex of roadway conditions. Even under these simple conditions, it was found that some drivers took over 3.5 s to respond. Because actual conditions on the highway are generally more complex than those of the studies, and because there is wide variation in driver reaction times, it is evident that the criterion adopted for use should be greater than 1.64 s. The brake reaction time used in design should be long enough to include the reaction times needed by nearly all drivers under most highway conditions. Both recent research (*17*) and the studies documented in the literature (*39*, *44*, *48*) show that a 2.5-s brake reaction time for stopping sight situations encompasses the capabilities of most drivers, including those of older drivers. The recommended design criterion of 2.5 s for brake reaction time exceeds the 90th percentile of reaction time for all drivers and was used in the development of Table 3-1.

A brake reaction time of 2.5 s is considered adequate for conditions that are more complex than the simple conditions used in laboratory and road tests, but it is not adequate for the most complex conditions encountered in actual driving. The need for greater reaction time in the most complex conditions encountered on the roadway, such as those found at multiphase at-grade intersections and at ramp terminals on through roadways, can be found in Section 3.2.3 on "Decision Sight Distance."

Braking Distance

The approximate braking distance of a vehicle on a level roadway traveling at the design speed of the roadway may be determined from the following equation:

Metric	U.S. Customary
$d_B = 0.039 \dfrac{V^2}{a}$	$d_B = 1.075 \dfrac{V^2}{a}$
where:	where:
d_B = braking distance, m	d_B = braking distance, ft
V = design speed, km/h	V = design speed, mph
a = deceleration rate, m/s^2	a = deceleration rate, ft/s^2

(3-1)

Studies documented in the literature (*17*) show that most drivers decelerate at a rate greater than 4.5 m/s^2 [14.8 ft/s^2] when confronted with the need to stop for an unexpected object in the roadway. Approximately 90 percent of all drivers decelerate at rates greater than 3.4 m/s^2 [11.2 ft/s^2]. Such decelerations are within the driver's capability to stay within his or her lane and maintain steering control during the braking maneuver on wet surfaces. Therefore, 3.4 m/s^2 [11.2 ft/s^2] (a comfortable deceleration for most drivers) is recommended as the deceleration threshold for determining stopping sight distance. Implicit in the choice of this deceleration threshold is the assessment that most vehicle braking systems and the tire-pavement friction levels of most roadways are capable of providing a deceleration rate of at least 3.4 m/s^2 [11.2 ft/s^2]. The friction available on most wet pavement surfaces and the capabilities of most vehicle braking systems can provide braking friction that exceeds this deceleration rate.

Table 3-1. Stopping Sight Distance on Level Roadways

Metric					U.S. Customary				
Design Speed (km/h)	Brake Reaction Distance (m)	Braking Distance on Level (m)	Stopping Sight Distance Calculat-ed (m)	Stopping Sight Distance Design (m)	Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance on Level (ft)	Stopping Sight Distance Calculat-ed (ft)	Stopping Sight Distance Design (ft)
20	13.9	4.6	18.5	20	15	55.1	21.6	76.7	80
30	20.9	10.3	31.2	35	20	73.5	38.4	111.9	115
40	27.8	18.4	46.2	50	25	91.9	60.0	151.9	155
50	34.8	28.7	63.5	65	30	110.3	86.4	196.7	200
60	41.7	41.3	83.0	85	35	128.6	117.6	246.2	250
70	48.7	56.2	104.9	105	40	147.0	153.6	300.6	305
80	55.6	73.4	129.0	130	45	165.4	194.4	359.8	360
90	62.6	92.9	155.5	160	50	183.8	240.0	423.8	425
100	69.5	114.7	184.2	185	55	202.1	290.3	492.4	495
110	76.5	138.8	215.3	220	60	220.5	345.5	566.0	570
120	83.4	165.2	248.6	250	65	238.9	405.5	644.4	645
130	90.4	193.8	284.2	285	70	257.3	470.3	727.6	730
					75	275.6	539.9	815.5	820
					80	294.0	614.3	908.3	910

Note: Brake reaction distance predicated on a time of 2.5 s; deceleration rate of 3.4 m/s² [11.2 ft/s²] used to determine calculated sight distance.

Design Values

The stopping sight distance is the sum of the distance traversed during the brake reaction time and the distance to brake the vehicle to a stop. The computed distances for various speeds at the assumed conditions on level roadways are shown in Table 3-1 and were developed from the following equation:

Metric

$$SSD = 0.278Vt + 0.039\frac{V^2}{a} \tag{3-2}$$

where:

SSD = stopping sight distance, m

V = design speed, km/h

t = brake reaction time, 2.5 s

a = deceleration rate, m/s²

U.S. Customary

$$SSD = 1.47Vt + 1.075\frac{V^2}{a} \tag{3-2}$$

where:

SSD = stopping sight distance, ft

V = design speed, mph

t = brake reaction time, 2.5 s

a = deceleration rate, ft/s²

Stopping sight distances exceeding those shown in Table 3-1 should be used as the basis for design wherever practical. Use of longer stopping sight distances increases the margin for error for all drivers and, in particular, for those who operate at or near the design speed during wet pavement conditions. New pavements should have initially, and should retain, friction coefficients consistent with the deceleration rates used to develop Table 3-1.

Effect of Grade on Stopping

When a highway is on a grade, Equation 3-1 for braking distance is modified as follows:

Metric	U.S. Customary
$d_B = \dfrac{V^2}{254\left[\left(\dfrac{a}{9.81}\right) \pm G\right]}$	$d_B = \dfrac{V^2}{30\left[\left(\dfrac{a}{32.2}\right) \pm G\right]}$ (3-3)
where:	where:
d_B = braking distance on grade, m	d_B = braking distance on grade, ft
V = design speed, km/h	V = design speed, mph
a = deceleration, m/s^2	a = deceleration, ft/s^2
G = grade, rise/run, m/m	G = grade, rise/run, ft/ft

In this equation, G is the rise in elevation divided by the distance of the run and the percent of grade divided by 100, and the other terms are as previously stated. The stopping distances needed on upgrades are shorter than on level roadways; those on downgrades are longer. The stopping sight distances for various grades shown in Table 3-2 are the values determined by using Equation 3-3 in place of the second term in Equation 3-2. These adjusted sight distance values are computed for wet-pavement conditions using the same design speeds and brake reaction times used for level roadways in Table 3-1.

Table 3-2. Stopping Sight Distance on Grades

Metric							U.S. Customary						
Design Speed (km/h)	Stopping Sight Distance (m)						Design Speed (mph)	Stopping Sight Distance (ft)					
	Downgrades			Upgrades				Downgrades			Upgrades		
	3 %	6 %	9 %	3 %	6 %	9 %		3 %	6 %	9 %	3 %	6 %	9 %
20	20	20	20	19	18	18	15	80	82	85	75	74	73
30	32	35	35	31	30	29	20	116	120	126	109	107	104
40	50	50	53	45	44	43	25	158	165	173	147	143	140
50	66	70	74	61	59	58	30	205	215	227	200	184	179
60	87	92	97	80	77	75	35	257	271	287	237	229	222
70	110	116	124	100	97	93	40	315	333	354	289	278	269
80	136	144	154	123	118	114	45	378	400	427	344	331	320
90	164	174	187	148	141	136	50	446	474	507	405	388	375
100	194	207	223	174	167	160	55	520	553	593	469	450	433
110	227	243	262	203	194	186	60	598	638	686	538	515	495
120	263	281	304	234	223	214	65	682	728	785	612	584	561
130	302	323	350	267	254	243	70	771	825	891	690	658	631
							75	866	927	1003	772	736	704
							80	965	1035	1121	859	817	782

On nearly all roads and streets, the grade is traversed by traffic in both directions of travel, but the sight distance at any point on the highway generally is different in each direction, particularly on straight roads in rolling terrain. As a general rule, the sight distance available on downgrades is larger than on upgrades, more or less automatically providing the appropriate corrections for grade. This may explain why some designers do not adjust stopping sight distance because of grade. Exceptions are one-way roadways or streets, as on divided highways with independent profiles. For these separate roadways, adjustments for grade may be needed.

Variation for Trucks

The recommended stopping sight distances are based on passenger car operation and do not explicitly consider design for truck operation. Trucks as a whole, especially the larger and heavier units, need longer stopping distances for a given speed than passenger vehicles. However, there is one factor that tends to balance the additional braking lengths for trucks with those for passenger cars. The truck driver is able to see substantially farther beyond vertical sight obstructions because of the higher position of the seat in the vehicle. Separate stopping sight distances for trucks and passenger cars, therefore, are not generally used in highway design.

There is one situation in which the goal should be to provide stopping sight distances greater than the design values in Table 3-1. Where horizontal sight restrictions occur on downgrades, particularly at the ends of long downgrades where truck speeds closely approach or exceed those of passenger cars, the greater height of eye of the truck driver is of little value. Although the average truck driver tends to be more experienced than the average passenger car driver and quicker to recognize potential risks, it is desirable under such conditions to provide stopping sight distance that exceeds the values in Tables 3-1 or 3-2.

3.2.3 Decision Sight Distance

Stopping sight distances are usually sufficient to allow reasonably competent and alert drivers to come to a hurried stop under ordinary circumstances. However, greater distances may be needed where drivers must make complex or instantaneous decisions, where information is difficult to perceive, or when unexpected or unusual maneuvers are needed. Limiting sight distances to those needed for stopping may preclude drivers from performing evasive maneuvers, which often involve less risk and are otherwise preferable to stopping. Even with an appropriate complement of standard traffic control devices in accordance with the *Manual on Uniform Traffic Control Devices* (MUTCD) (*22*), stopping sight distances may not provide sufficient visibility distances for drivers to corroborate advance warning and to perform the appropriate maneuvers. It is evident that there are many locations where it would be prudent to provide longer sight distances. In these circumstances, decision sight distance provides the greater visibility distance that drivers need.

Decision sight distance is the distance needed for a driver to detect an unexpected or otherwise difficult-to-perceive information source or condition in a roadway environment that may be visually cluttered, recognize the condition or its potential threat, select an appropriate speed and path, and initiate and complete complex maneuvers (*9*). Because decision sight distance offers drivers additional margin for error and affords them sufficient length to maneuver their vehicles at the same or reduced speed, rather than to just stop, its values are substantially greater than stopping sight distance.

Drivers need decision sight distances whenever there is likelihood for error in either information reception, decision making, or control actions (*40*). Examples of critical locations where these kinds of errors are likely to occur, and where it is desirable to provide decision sight distance include interchange and intersection locations where unusual or unexpected maneuvers are needed, changes in cross section such as toll plazas and lane drops, and areas of concentrated demand where there is apt to be "visual noise" from competing sources of information, such as roadway elements, traffic, traffic control devices, and advertising signs.

The decision sight distances in Table 3-3 may be used to (1) provide values for sight distances that may be appropriate at critical locations, and (2) serve as criteria in evaluating the suitability of the available sight distances at these locations. Because of the additional maneuvering space provided, decision sight distances should be considered at critical locations or critical decision points should be moved to locations where sufficient decision sight distance is available. If it is not practical to provide decision sight distance because of horizontal or vertical curvature or if relocation of decision points is not practical, special attention should be given to the use of suitable traffic control devices for providing advance warning of the conditions that are likely to be encountered.

Table 3-3. Decision Sight Distance

Metric						U.S. Customary					
Design Speed (km/h)	Decision Sight Distance (m)					Design Speed (mph)	Decision Sight Distance (ft)				
	Avoidance Maneuver						Avoidance Maneuver				
	A	B	C	D	E		A	B	C	D	E
50	70	155	145	170	195	30	220	490	450	535	620
60	95	195	170	205	235	35	275	590	525	625	720
70	115	325	200	235	275	40	330	690	600	715	825
80	140	280	230	270	315	45	395	800	675	800	930
90	170	325	270	315	360	50	465	910	750	890	1030
100	200	370	315	355	400	55	535	1030	865	980	1135
110	235	420	330	380	430	60	610	1150	990	1125	1280
120	265	470	360	415	470	65	695	1275	1050	1220	1365
130	305	525	390	450	510	70	780	1410	1105	1275	1445
						75	875	1545	1180	1365	1545
						80	970	1685	1260	1455	1650

Avoidance Maneuver A: Stop on rural road—t = 3.0 s

Avoidance Maneuver B: Stop on urban road—t = 9.1 s

Avoidance Maneuver C: Speed/path/direction change on rural road—t varies between 10.2 and 11.2 s

Avoidance Maneuver D: Speed/path/direction change on suburban road—t varies between 12.1 and 12.9 s

Avoidance Maneuver E: Speed/path/direction change on urban road—t varies between 14.0 and 14.5 s

Decision sight distance criteria that are applicable to most situations have been developed from empirical data. The decision sight distances vary depending on whether the location is on a rural or urban road and on the type of avoidance maneuver needed to negotiate the location properly. Table 3-3 shows decision sight distance values for various situations rounded for design. As can be seen in the table, shorter distances are generally needed for rural roads and for locations where a stop is the appropriate maneuver.

For the avoidance maneuvers identified in Table 3-3, the pre-maneuver time is greater than the brake reaction time for stopping sight distance to allow the driver additional time to detect and recognize the roadway or traffic situation, identify alternative maneuvers, and initiate a response at critical locations on the highway (*45*). The pre-maneuver component of decision sight distance uses a value ranging between 3.0 and 9.1 s (*51*).

The braking distance for the design speed is added to the pre-maneuver component for avoidance maneuvers A and B as shown in Equation 3-4. The braking component is replaced in avoidance maneuvers C, D, and E with a maneuver distance based on maneuver times, between 3.5 and 4.5 s, that decrease with increasing speed (*45*) in accordance with Equation 3-5.

The decision sight distances for avoidance maneuvers A and B are determined as:

Metric	U.S. Customary	
$DSD = 0.278Vt + 0.039\frac{V^2}{a}$	$DSD = 1.47Vt + 1.075\frac{V^2}{a}$	(3-4)
where:	where:	
DSD = decision sight distance, m	DSD = decision sight distance, ft	
t = pre-maneuver time, s (see notes in Table 3-3)	t = pre-maneuver time, s (see notes in Table 3-3)	
V = design speed, km/h	V = design speed, mph	
a = driver deceleration, m/s^2	a = driver deceleration, ft/s^2	

The decision sight distances for avoidance maneuvers C, D, and E are determined as:

Metric	U.S. Customary	
$DSD = 0.278Vt$	$DSD = 1.47Vt$	(3-5)
where:	where:	
DSD = decision sight distance, m	DSD = decision sight distance, ft	
t = total pre-maneuver and maneuver time, s (see notes in Table 3-3)	t = total pre-maneuver and maneuver time, s (see notes in Table 3-3)	
V = design speed, km/h	V = design speed, mph	

3.2.4 Passing Sight Distance for Two-Lane Highways

Criteria for Design

Most roads and many streets are two-lane, two-way highways on which vehicles frequently overtake slower moving vehicles. Passing maneuvers in which faster vehicles move ahead of slower vehicles are accomplished on lanes regularly used by opposing traffic. If passing is to be accomplished without interfering with an opposing vehicle, the passing driver should be able to see a sufficient distance ahead, clear of traffic, so the passing driver can decide whether to initiate and to complete the passing maneuver without cutting off the passed vehicle before meeting an opposing vehicle that appears during the maneuver. When appropriate, the driver can return to the right lane without completing the pass if he or she sees opposing traffic is too close when the maneuver is only partially completed. Many passing maneuvers are accomplished without the driver being able to see any potentially conflicting vehicle at the beginning of the maneuver. An alternative to providing passing sight distance is found in Section 3.4.4 under "Passing Lanes."

Minimum passing sight distances for use in design are based on the minimum sight distances presented in the MUTCD (*22*) as warrants for no-passing zones on two-lane highways. Design practice should be most effective when it anticipates the traffic controls (i.e., passing and no-passing zone markings) that will be placed on the highways. The potential for conflicts in passing operations on two-lane highways is ultimately determined by the judgments of drivers that initiate and complete passing maneuvers in response to (1) the driver's view of the road ahead as provided by available passing sight distance and (2) the passing and no-passing zone markings. Recent research has shown that the MUTCD passing sight distance criteria result in two-lane highways that experience very few crashes related to passing maneuvers (*20*, *34*).

Design Values

The design values for passing sight distance are presented in Table 3-4 and are shown in comparison to stopping sight distance criteria in Figure 3-1. It is apparent from the comparison in Figure 3-1 that more sight distance is needed to accommodate passing maneuvers on a two-lane highway than for stopping sight distance that is provided continuously along the highway.

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

Metric				U.S. Customary			
Design Speed (km/h)	Assumed Speeds (km/h)		Passing Sight Distance (m)	Design Speed (mph)	Assumed Speeds (mph)		Passing Sight Distance (ft)
	Passed Vehicle	Passing Vehicle			Passed Vehicle	Passing Vehicle	
30	11	30	120	20	8	20	400
40	21	40	140	25	13	25	450
50	31	50	160	30	18	30	500
60	41	60	180	35	23	35	550
70	51	70	210	40	28	40	600
80	61	80	245	45	33	45	700
90	71	90	280	50	38	50	800
100	81	100	320	55	43	55	900
110	91	110	355	60	48	60	1000
120	101	120	395	65	53	65	1100
130	111	130	440	70	58	70	1200
				75	63	75	1300
				80	68	80	1400

Research has verified that the passing sight distance values in Table 3-4 are consistent with field observation of passing maneuvers (*34*). This research used two theoretical models for the sight distance needs of passing drivers; both models were based on the assumption that a passing driver will abort the passing maneuver and return to his or her normal lane behind the passed vehicle if a potentially conflicting vehicle comes into view before reaching a critical position in the passing maneuver beyond which the passing driver is committed to complete the maneuver. The Glennon model (*26*) assumes that the critical position occurs where the passing sight distance to complete the maneuver is equal to the sight distance needed to abort the maneuver. The Hassan et al. model (*35*) assumes that the critical position occurs where the passing sight distances to complete or abort the maneuver are equal or where the passing and passed vehicles are abreast, whichever occurs first.

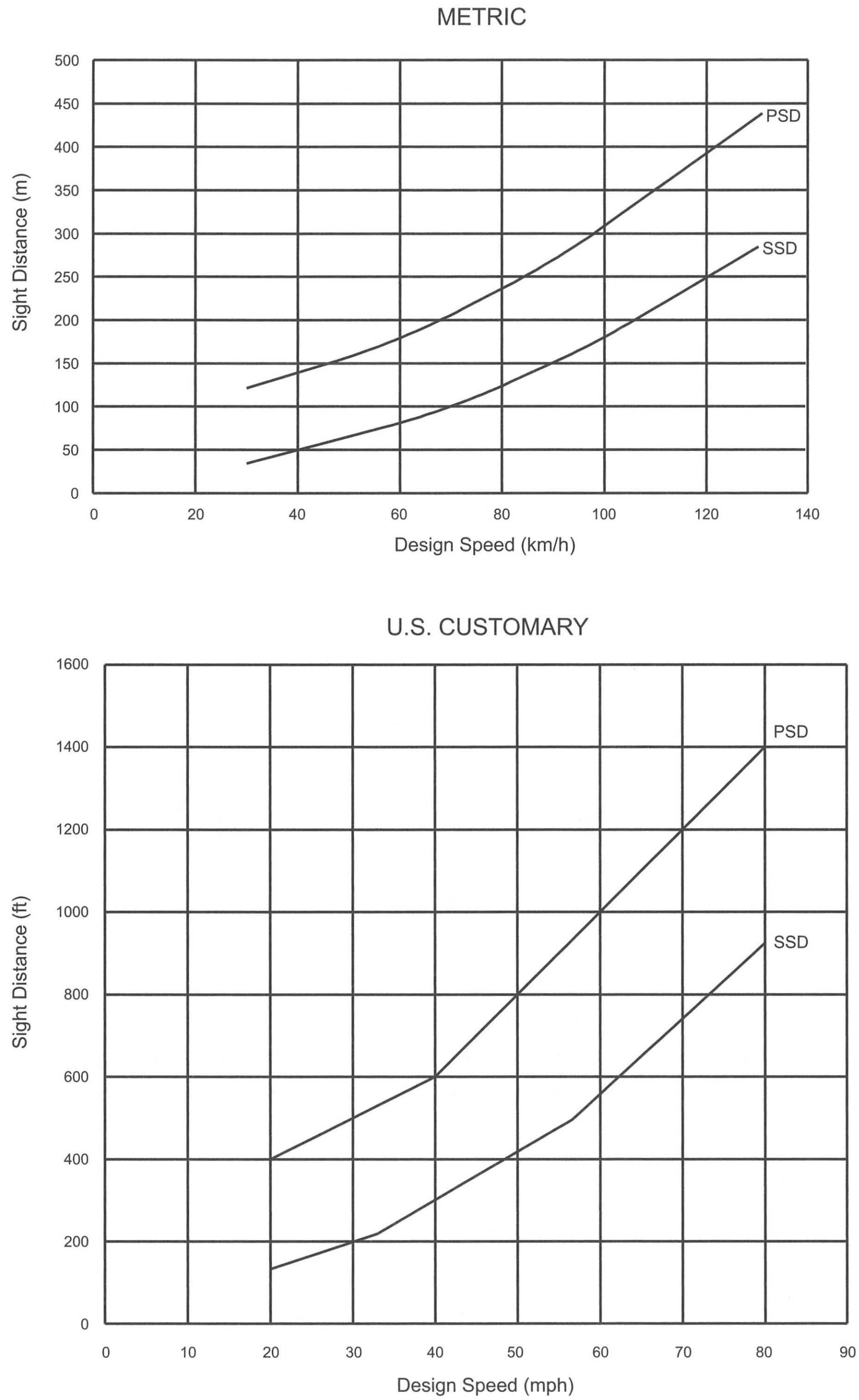


Figure 3-1. Comparison of Design Values for Passing Sight Distance and Stopping Sight Distance

Minimum passing sight distances for design of two-lane highways incorporate certain assumptions about driver behavior. Actual driver behavior in passing maneuvers varies widely. To accommodate these variations in driver behavior, the design criteria for passing sight distance should accommodate the behavior of a high percentage of drivers, rather than just the average driver. The assumptions made in applying the Glennon and Hassan et al. models (*25*, *35*) are as follows:

1. The speeds of the passing and opposing vehicles are equal and represent the design speed of the highway.
2. The passed vehicle travels at uniform speed and speed differential between the passing and passed vehicles is 19 km/h [12 mph].
3. The passing vehicle has sufficient acceleration capability to reach the specified speed differential relative to the passed vehicle by the time it reaches the critical position, which generally occurs about 40 percent of the way through the passing maneuver.
4. The lengths of the passing and passed vehicles are 5.8 m [19 ft], as shown for the PC design vehicle in Section 2.1.1.
5. The passing driver's perception-reaction time in deciding to abort passing a vehicle is 1 s.
6. If a passing maneuver is aborted, the passing vehicle will use a deceleration rate of 3.4 m/s^2 [11.2 ft/s^2], the same deceleration rate used in stopping sight distance criteria.
7. For a completed or aborted pass, the space headway between the passing and passed vehicles is 1 s.
8. The minimum clearance between the passing and opposed vehicles at the point at which the passing vehicle returns to its normal lane is 1 s.

The application of the passing sight distance models using these assumptions is presented in NCHRP Report 605 (*34*).

The passing sight distance for use in design should be based on a single passenger vehicle passing a single passenger vehicle. While there may be occasions to consider multiple passings, where two or more vehicles pass or are passed, it is not practical to assume such conditions in developing minimum design criteria. Research has shown that longer sight distances are often needed for passing maneuvers when the passed vehicle, the passing vehicle, or both are trucks (*30*). Longer sight distances occur in design, and such locations can accommodate an occasional multiple passing maneuver or a passing maneuver involving a truck.

Frequency and Length of Passing Sections

Sight distance adequate for passing should be encountered frequently on two-lane highways. Each passing section along a length of roadway with sight distance ahead equal to or greater than the minimum passing sight distance should be as long as practical. The frequency and length of passing sections for highways principally depend on the topography, the design speed of highway, and the cost. For streets, the spacing of intersections is the principal consideration.

It is not practical to directly indicate the frequency with which passing sections should be provided on two-lane highways due to the physical constraints and cost limitations. During the course of normal design, passing sections are provided on almost all highways and selected streets, but the designer's appreciation of their importance and a studied attempt to provide them can usually enable others to be provided at little or no additional cost. In steep mountainous terrain, it may be more economical to build intermittent four-lane sections or passing lanes with stopping sight distance on some two-lane highways, in lieu of two-lane sections with passing sight distance. Alternatives are discussed in "Passing Lanes" of Section 3.4.4.

The passing sight distances shown in Table 3-4 are sufficient for a single or isolated pass only. Designs with infrequent passing sections may not provide enough passing opportunities for efficient traffic operations. Even on low-volume roadways, a driver desiring to pass may, on reaching the passing section, find vehicles in the opposing lane and thus be unable to use the passing section or at least may not be able to begin to pass at once.

The importance of frequent passing sections is illustrated by their effect on the level of service of a two-lane, two-way highway. The procedures in the *Highway Capacity Manual* (HCM) (*62*) to analyze two-lane, two-way highways base the level-of-service criteria on two measures of effectiveness—percent time spent following and average travel speed. Both of these criteria are affected by the lack of passing opportunities. The HCM procedures show, for example, up to a 19 percent increase in the percent time spent following when the directional split is 50/50 and no-passing zones comprise 40 percent of the analysis length compared to a highway with similar traffic volumes and no sight restrictions. The effect of restricted passing sight distance is even more severe for unbalanced flow and where the no-passing zones comprise more than 40 percent of the length.

There is a similar effect on the average travel speed. As the percent of no-passing zones increases, there is an increased reduction in the average travel speed for the same demand flow rate. For example, a demand flow rate of 800 passenger cars per hour incurs a reduction of 3.1 km/h [1.9 mph] when no-passing zones comprise 40 percent of the analysis length compared to no reduction in speed on a route with unrestricted passing.

The HCM procedures indicate another possible criterion for passing sight distance design on two-lane highways that are several miles or more in length. The available passing sight distances along this length can be summarized to show the percentage of length with greater-than-minimum passing sight distance. Analysis of capacity related to this percentage would indicate whether or not alignment and profile adjustments are needed to accommodate the design hourly volume (DHV). When highway sight distances are analyzed over the whole range of lengths within which passing maneuvers are made, a new design criterion may be evaluated. Where high traffic volumes are expected on a highway and a high level of service is to be maintained, frequent or nearly continuous passing sight distances should be provided.

The HCM procedures and other traffic models can be used in design to determine the level of service that will be provided by the passing sight distance profile for any proposed design alternative. The level of service provided by the proposed design should be compared to the highway agency's desired level of service for the project and, if the desired level of service is not achieved, the feasibility and practicality of adjustments to the design to provide additional passing sight distance should be considered. Passing sections shorter than 120 to 240 m [400 to 800 ft] have been found to contribute little to improving the traffic operational efficiency of a two-lane highway. In determining the percentage of roadway length with greater-than-minimum passing sight distance, passing sections shorter than the minimum lengths shown in Table 3-5 should be excluded from consideration.

Table 3-5. Minimum Passing Zone Lengths to Be Included in Traffic Operational Analyses

Metric		U.S. Customary	
85th Percentile Speed or Posted or Statutory Speed Limit (km/h)	Minimum Passing Zone Length (m)	85th Percentile Speed or Posted or Statutory Speed Limit (mph)	Minimum Passing Zone Length (ft)
40	140	20	400
50	180	30	550
60	210	35	650
70	240	40	750
80	240	45	800
90	240	50	800
100	240	55	800
110	240	60	800
120	240	65	800
		70	800

3.2.5 Sight Distance for Multilane Highways

There is no need to consider passing sight distance on highways or streets that have two or more traffic lanes in each direction of travel. Passing maneuvers on multilane roadways are expected to occur within the limits of the traveled way for each direction of travel. Thus, passing maneuvers that involve crossing the centerline of four-lane undivided roadways or crossing the median of four-lane roadways should be prohibited.

Multilane roadways should have continuously adequate stopping sight distance, with greater-than-design sight distances preferred. Design criteria for stopping sight distance vary with vehicle speed and are discussed in detail in Section 3.2.2 on "Stopping Sight Distance."

3.2.6 Criteria for Measuring Sight Distance

Sight distance is the distance along a roadway throughout which an object of specified height is continuously visible to the driver. This distance is dependent on the height of the driver's eye above the road surface, the specified object height above the road surface, and the height and lateral position of sight obstructions within the driver's line of sight.

Height of Driver's Eye

For all sight distance calculations for passenger vehicles, the height of the driver's eye is considered to be 1.08 m [3.50 ft] above the road surface. This value is based on a study (*17*) that found average vehicle heights have decreased to 1.30 m [4.25 ft] with a comparable decrease in average eye heights to 1.08 m [3.50 ft]. Because of various factors that appear to place practical limits on further decreases in passenger car heights and the relatively small increases in the lengths of vertical curves that would result from further changes that do occur, 1.08 m [3.50 ft] is considered to be the appropriate height of driver's eye for measuring both stopping and passing sight distances. For large trucks, the driver eye height ranges from 1.80 to 2.40 m [3.50 to 7.90 ft]. The recommended value of truck driver eye height for design is 2.33 m [7.60 ft] above the road surface.

Height of Object

For stopping sight distance and decision sight distance calculations, the height of object is considered to be 0.60 m [2.00 ft] above the road surface. For passing sight distance calculations, the height of object is considered to be 1.08 m [3.50 ft] above the road surface.

Stopping sight distance object—The selection of a 0.60-m [2.00-ft] object height was based on research indicating that objects with heights less than 0.60 m [2.00 ft] are seldom involved in crashes (*17*). Therefore, it is considered that an object 0.60 m [2.00 ft] in height is representative of the smallest object that involves risk to drivers. An object height of 0.60 m [2.00 ft] is representative of the height of automobile headlights and taillights. Using object heights of less than 0.60 m [2.00 ft] for stopping sight distance calculations would result in longer crest vertical curves without a documented decrease in the frequency or severity of crashes (*17*). Object height of less than 0.60 m [2.00 ft] could substantially increase construction costs because additional excavation would be needed to provide the longer crest vertical curves. It is also doubtful that the driver's ability to perceive situations involving risk of collisions would be increased because recommended stopping sight distances for high-speed design are beyond most drivers' capabilities to detect objects less than 0.60 m [2.00 ft] in height (*17*).

Passing sight distance object—An object height of 1.08 m [3.50 ft] is adopted for passing sight distance. This object height is based on a vehicle height of 1.33 m [4.35 ft], which represents the 15th percentile of vehicle heights in the current passenger car population, less an allowance of 0.25 m [0.85 ft], which represents a near-maximum value for the portion of the vehicle height that needs to be visible for another driver to recognize a vehicle as such (*32*). Passing sight distances calculated on this basis are also considered adequate for night conditions because headlight beams of an opposing vehicle generally can be seen from a greater distance than a vehicle can be recognized in the daytime. The choice of an object height equal to the driver eye height makes passing sight distance design reciprocal (i.e., when the driver of the passing vehicle can see the opposing vehicle, the driver of the opposing vehicle can also see the passing vehicle).

Intersection sight distance object—As in the case of passing sight distance, the object to be seen by the driver in an intersection sight distance situation is another vehicle. Therefore, design for intersection sight distance is based on the same object height used in design for passing sight distance, 1.08 m [3.50 ft].

Decision sight distance object—The 0.60-m [2.00-ft] object-height criterion adopted for stopping sight distance is also used for decision sight distance. The rationale for applying this object height for decision sight distance is the same as for stopping sight distance.

Sight Obstructions

On a tangent roadway, the obstruction that limits the driver's sight distance is the road surface at some point on a crest vertical curve. On horizontal curves, the obstruction that limits the driver's sight distance may be the road surface at some point on a crest vertical curve or it may be some physical feature outside of the traveled way, such as a longitudinal barrier, a bridge-approach fill slope, a tree, foliage, or the backslope of a cut section. Accordingly, all highway construction plans should be checked in both the vertical and horizontal plane for sight distance obstructions.

Measuring and Recording Sight Distance

The design of horizontal alignment and vertical profile using sight distance and other criteria is addressed in Sections 3.3 through 3.5, including the detailed design of horizontal and vertical curves. Sight distance should be considered in the preliminary stages of design when both the horizontal and vertical alignment are still subject to adjustment. By determining the available sight distances graphically on the plans and recording them at frequent intervals, the designer can review the overall layout and produce a more balanced design by minor adjustments in the plan or profile. Methods for scaling sight distances on plans are demonstrated in Figure 3-2, which also shows a typical sight distance record that would be shown on the final plans.

Because the view of the highway ahead may change rapidly in a short travel distance, it is desirable to measure and record sight distance for both directions of travel at each station. Both horizontal and vertical sight distances should be measured and the shorter lengths recorded. In the case of a two-lane highway, passing sight distance should be measured and recorded in addition to stopping sight distance.

Sight distance information, such as that presented in Figures 3-41 and 3-43, may be used to establish minimum lengths of vertical curves. Charts similar to Table 3-28 are useful for determining the radius of horizontal curve or the lateral offset from the traveled way needed to provide the design sight distance. Examining sight distances along the proposed highway may be accomplished by direct scaling. Sight distance can be easily determined where plans and profiles are drawn using computer-aided design and drafting (CADD) systems. The following discussion presents a method for scaling sight distances.

Horizontal sight distance on the inside of a curve is limited by obstructions such as buildings, hedges, wooded areas, high ground, or other topographic features. These are generally plotted on the plans. Horizontal sight is measured with a straightedge, as indicated in the upper left portion of Figure 3-2. The cut slope obstruction is shown on the worksheets by a line representing the proposed excavation slope at a point 0.84 m [2.75 ft] above the road surface (i.e., the approximate average of 1.08 and 0.60 m [3.50 and 2.00 ft] for stopping sight distance and a point about 1.080 m [3.50 ft] above the road surface for passing sight distance. The position of this line with respect to the centerline may be scaled from the plotted highway cross sections. Preferably, the stopping sight distance should be measured between points on one traffic lane and passing sight distance from the middle of the other lane.

Such refinement on two-lane highways generally is not needed and measurement of sight distance along the centerline or traveled-way edge is suitable. Where there are changes of grade coincident with horizontal curves that have sight-limiting cut slopes on the inside, the line-of-sight intercepts the slope at a level either lower or higher than the assumed average height. In measuring sight distance, the error in use of the assumed 0.84- or 1.08-m [2.75- or 3.50-ft] height usually can be ignored.

Vertical sight distance may be scaled from a plotted profile by the method illustrated at the right center of Figure 3-2. A transparent strip with parallel edges 1.08 m [3.50 ft] apart and with a scratched line 0.60 m [2.00 ft] from the upper edge, in accordance with the vertical scale, is a useful tool. The lower edge of the strip is placed on the station from which the vertical sight distance is desired, and the strip is pivoted about this point until the upper edge is tangent to the profile. The distance between the initial station and the station on the profile intersected by the 0.60-m [2.00-ft] line is the stopping sight distance. The distance between the initial station and the station on the profile intersected by the lower edge of the strip is the passing sight distance.

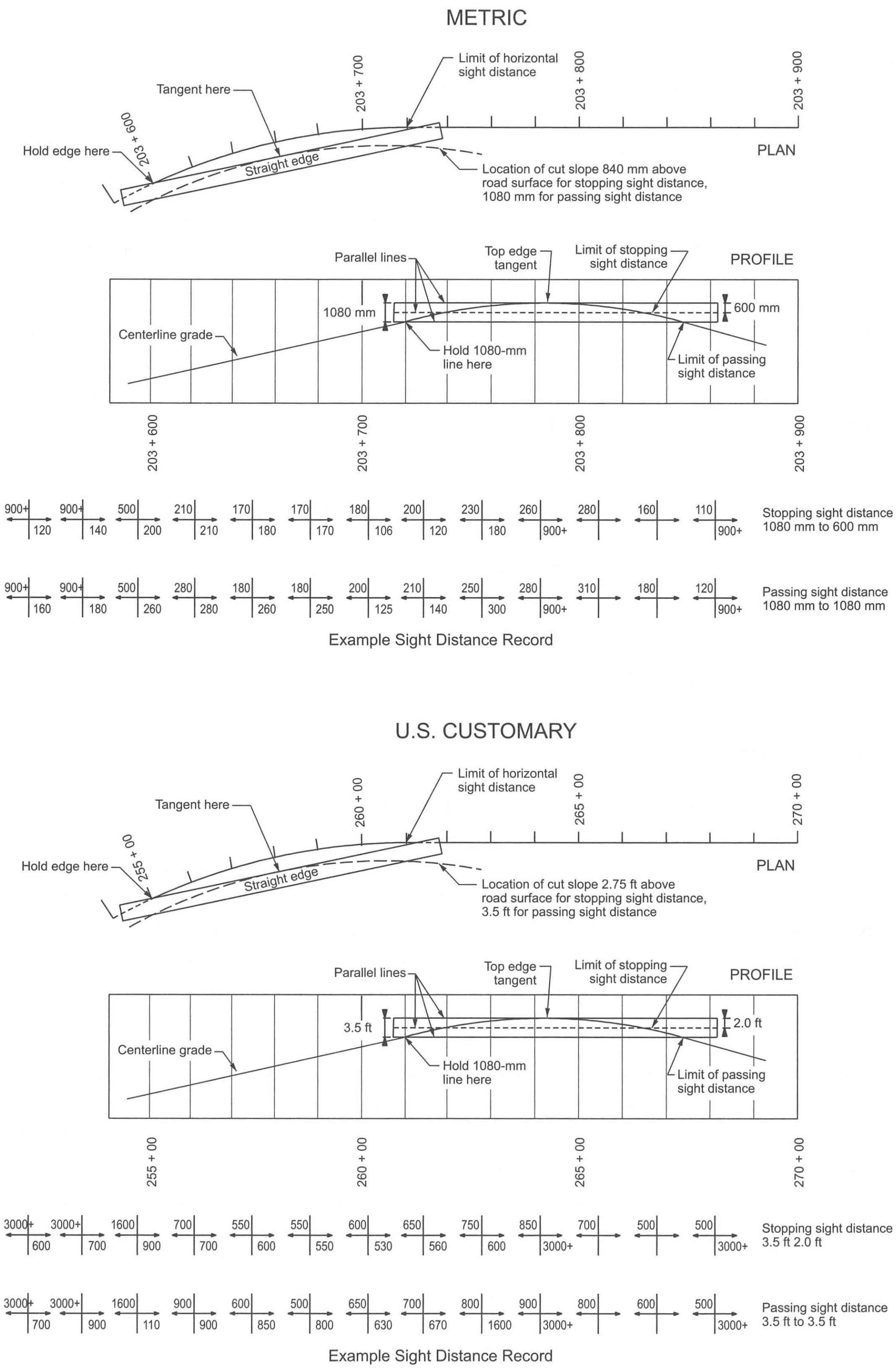


Figure 3-2. Scaling and Recording Sight Distances on Plans

A simple sight distance record is shown in the lower part of Figure 3-2. Sight distances in both directions are indicated by arrows and figures at each station on the plan and profile sheet of the proposed highway. To avoid the extra work of measuring unusually long sight distances that may occasionally be found, a selected maximum value may be recorded. In the example shown, all sight distances of more than 1 000 m [3,000 ft] are recorded as 1 000 m+ [3,000 ft+], and where this occurs for several consecutive stations, the intermediate values are omitted. Sight distances less than 500 m [1,500 ft] may be scaled to the nearest 10 m [50 ft] and those greater than 500 m [1,500 ft] to the nearest 50 m [100 ft]. The available sight distances along a proposed highway also may be shown by other methods. Several states use a sight distance graph, plotted in conjunction with the plan and profile of the highway, as a means of demonstrating sight distances.

Sight distance records for two-lane highways may be used effectively to tentatively determine the marking of no-passing zones in accordance with criteria given in the MUTCD (*22*). Marking of such zones is an operational rather than a design responsibility. No-passing zones thus established serve as a guide for markings when the highway is completed. The zones so determined should be checked and adjusted by field measurements before actual markings are placed.

Sight distance records also are useful on two-lane highways for determining the percentage of length of highway on which sight distance is restricted to less than the passing minimum, which is important in evaluating capacity. With recorded sight distances, as in the lower part of Figure 3-2, it is a simple process to determine the percentage of length of highway with a given sight distance or greater.

3.3 HORIZONTAL ALIGNMENT

3.3.1 Theoretical Considerations

To achieve balance in highway design, all geometric elements should, as far as economically practical, be designed to operate at a speed likely to be observed under the normal conditions for that roadway for a vast majority of motorists. Generally, this can be achieved through the use of design speed as an overall design control. The design of roadway curves should be based on an appropriate relationship between design speed and curvature and on their joint relationships with superelevation (roadway banking) and side friction. Although these relationships stem from the laws of mechanics, the actual values for use in design depend on practical limits and factors determined more or less empirically. These limits and factors are explained in the following discussion.

When a vehicle moves in a circular path, it undergoes a centripetal acceleration that acts toward the center of curvature. This acceleration is sustained by a component of the vehicle's weight related to the roadway superelevation, by the side friction developed between the vehicle's tires and the pavement surface, or by a combination of the two. Centripetal acceleration is sometimes equated to centrifugal force. However, this is an imaginary force that motorists believe is pushing them outward while cornering when, in fact, they are truly feeling the vehicle being accelerated in an inward direction. In horizontal curve design, "lateral acceleration" is equivalent to "centripetal acceleration"; the term "lateral acceleration" is used in this policy as it is specifically applicable to geometric design.

From the laws of mechanics, the basic equation that governs vehicle operation on a curve is:

Metric	U.S. Customary
$\frac{0.01e+f}{1-0.01ef}=\frac{v^2}{gR}=\frac{0.0079V^2}{R}=\frac{V^2}{127R}$	$\frac{0.01e+f}{1-0.01ef}=\frac{v^2}{gR}=\frac{0.067V^2}{R}=\frac{V^2}{15R}$ (3-6)
where:	where:
e = rate of roadway superelevation, percent	e = rate of roadway superelevation, percent
f = side friction (demand) factor	f = side friction (demand) factor
v = vehicle speed, m/s	v = vehicle speed, ft/s
g = gravitational constant, 9.81 m/s^2	g = gravitational constant, 32.2 ft/s^2
V = vehicle speed, km/h	V = vehicle speed, mph
R = radius of curve measured to a vehicle's center of gravity, m	R = radius of curve measured to a vehicle's center of gravity, ft

Equation 3-6, which models the moving vehicle as a point mass, is often referred to as the basic curve equation.

When a vehicle travels at constant speed on a curve superelevated so that the f value is zero, the centripetal acceleration is sustained by a component of the vehicle's weight and, theoretically, no steering force is needed. A vehicle traveling faster or slower than the balance speed develops tire friction as steering effort is applied to prevent movement to the outside or to the inside of the curve. On nonsuperelevated curves, travel at different speeds is also possible by utilizing appropriate amounts of side friction to sustain the varying lateral acceleration.

3.3.2 General Considerations

From accumulated research and experience, limiting values for superelevation rate (e_{max}) and side friction demand (f_{max}) have been established for curve design. Using these established limiting values in the basic curve formula permits determining a minimum curve radius for various design speeds. Use of curves with radii larger than this minimum allows superelevation, side friction, or both to have values below their respective limits. The amount by which each factor is below its respective limit is chosen to provide an equitable contribution of each factor toward sustaining the resultant lateral acceleration. The methods used to achieve this equity for different design situations are discussed below.

Superelevation

There are practical upper limits to the rate of superelevation on a horizontal curve. These limits relate to considerations of climate, constructability, adjacent land use, and the frequency of slow-moving vehicles. Where snow and ice are a factor, the rate of superelevation should not exceed the rate on which vehicles standing or traveling slowly would slide toward the center of the curve when the pavement is icy. At higher speeds, the phenomenon of partial hydroplaning can occur on curves with poor drainage that allows water to build up on the pavement surface. Skidding occurs, usually at the rear wheels, when the lubricating effect of the water film reduces the available lateral friction below the friction demand for cornering. When travelling slowly around a curve with high superelevation, negative lateral forces develop and the vehicle is held in the proper path only when the driver steers up the slope or against the direction of the horizontal curve. Steering in this direction seems unnatural to the driver and may explain the difficulty of driving on roads where the superelevation is in excess of that needed for travel at normal speeds. Such high rates of superelevation are undesirable on high-volume roads, as in urban and suburban areas, where there are numerous occasions when vehicle speeds should be substantially reduced because of the volume of traffic or other conditions.

Some vehicles have high centers of gravity and some passenger cars are loosely suspended on their axles. When these vehicles travel slowly on steep cross slopes, the down-slope tires carry a high percentage of the vehicle weight. A vehicle can roll over if this condition becomes extreme.

A discussion of these considerations and the rationale used to establish an appropriate maximum rate of superelevation for design of horizontal curves is provided in the subsection on "Maximum Superelevation Rates for Streets and Highways" in Section 3.3.3.

Side Friction Factor

The side friction factor represents the vehicle's need for side friction, also called the side friction demand; it also represents the lateral acceleration a_f that acts on the vehicle. This acceleration can be computed as the product of the side friction demand factor f and the gravitational constant g (i.e., $a_f = f_g$). Note that the lateral acceleration actually experienced by vehicle occupants tends to be slightly larger than predicted by the product f_g due to vehicle body roll angle.

With the wide variation in vehicle speeds on curves, there usually is an unbalanced force whether the curve is superelevated or not. This force results in tire side thrust, which is counterbalanced by friction between the tires and the pavement surface. This frictional counterforce is developed by distortion of the contact area of the tire.

The coefficient of friction f is the friction force divided by the component of the weight perpendicular to the pavement surface and is expressed as a simplification of the basic curve formula shown as Equation 3-6. The value of the product ef in this formula is always small. As a result, the $1 - 0.01ef$ term is nearly equal to 1.0 and is normally omitted in highway design. Omission of this term yields the following basic side friction equation:

Metric	U.S. Customary	
$f = \dfrac{V^2}{127R} - 0.01e$	$f = \dfrac{V^2}{15R} - 0.01e$	(3-7)

This equation is referred to as the simplified curve formula and yields slightly larger (and, thus, more conservative) estimates of friction demand than would be obtained using the basic curve formula.

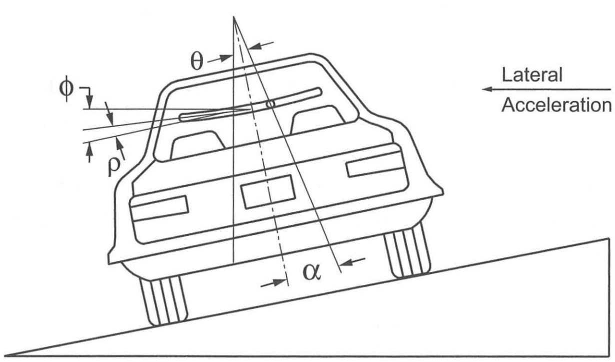
The coefficient f has been called lateral ratio, cornering ratio, unbalanced centrifugal ratio, friction factor, and side friction factor. Because of its widespread use, the term "side friction factor" is used in this discussion. The upper limit of the side friction factor is the point at which the tire would begin to skid; this is known as the point of impending skid. Because highway curves are designed so vehicles can avoid skidding with a margin of safety, the f values used in design should be substantially less than the coefficient of friction at impending skid.

The side friction factor at impending skid depends on a number of other factors, among which the most important are the speed of the vehicle, the type and condition of the roadway surface, and the type and condition of the vehicle tires. Different observers have recorded different maximum side friction factors at the same speeds for pavements of similar composition, and logically so, because of the inherent variability in pavement texture, weather conditions, and tire condition. In general, studies show that the maximum side friction factors developed between new tires and wet concrete pavements range from about 0.5 at 30 km/h [20 mph] to approximately 0.35 at 100 km/h [60 mph]. For normal wet concrete pavements and smooth tires, the maximum side friction factor at impending skid is about 0.35 at 70 km/h [45 mph]. In all cases, the studies show a decrease in friction values as speeds increase (*46*, *47*, *60*).

Horizontal curves should not be designed directly on the basis of the maximum available side friction factor. Rather, the maximum side friction factor used in design should be that portion of the maximum available side friction that can be used with comfort, and without likelihood of skidding, by the vast majority of drivers. Side friction levels that represent pavements that are glazed, bleeding, or otherwise lacking in reasonable skid-resistant properties should not control design because such conditions are avoidable and geometric design should be based on acceptable surface conditions attainable at reasonable cost.

A key consideration in selecting maximum side friction factors for use in design is the level of lateral acceleration that is sufficient to cause drivers to experience a feeling of discomfort and to react instinctively to avoid higher speed. The speed on a curve at which discomfort due to the lateral acceleration is evident to drivers is used as a design control for the maximum side friction factor on high-speed streets and highways. At low speeds, drivers are more tolerant of discomfort, thus permitting employment of an increased amount of side friction for use in design of horizontal curves.

The ball-bank indicator has been widely used by research groups, local agencies, and highway departments as a uniform measure of lateral acceleration to set speeds on curves that avoid driver discomfort. It consists of a steel ball in a sealed glass tube; except for the damping effect of the liquid in the tube, the ball is free to roll. Its simplicity of construction and operation has led to widespread acceptance as a guide for determination of appropriate curve speeds. With such a device mounted in a vehicle in motion, the ball-bank reading at any time is indicative of the combined effect of body roll, lateral acceleration angle, and superelevation as shown in Figure 3-3.



α = Ball-bank indicator angle
ϕ = Body roll angle
θ = Superelevation angle
ρ = Lateral acceleration angle

Figure 3-3. Geometry for Ball-Bank Indicator

The lateral acceleration developed as a vehicle travels at uniform speed on a curve causes the ball to roll out to a fixed angle position as shown in Figure 3-3. A correction should be made for that portion of the force taken up in the small body-roll angle. The indicated side force perceived by the vehicle occupants is thus on the order of $F \approx \tan(\alpha - \rho)$.

In a series of definitive tests (*47*), it was concluded that speeds on curves that avoid driver discomfort are indicated by ball-bank readings of 14 degrees for speeds of 30 km/h [20 mph] or less, 12 degrees for speeds of 40 and 50 km/h [25 and 30 mph], and 10 degrees for speeds of 55 through 80 km/h [35 through 50 mph]. These ball-bank readings are indicative of side friction factors of 0.21, 0.18, and 0.15, respectively, for the test body roll angles and provide ample margin of safety against skidding or vehicle rollover.

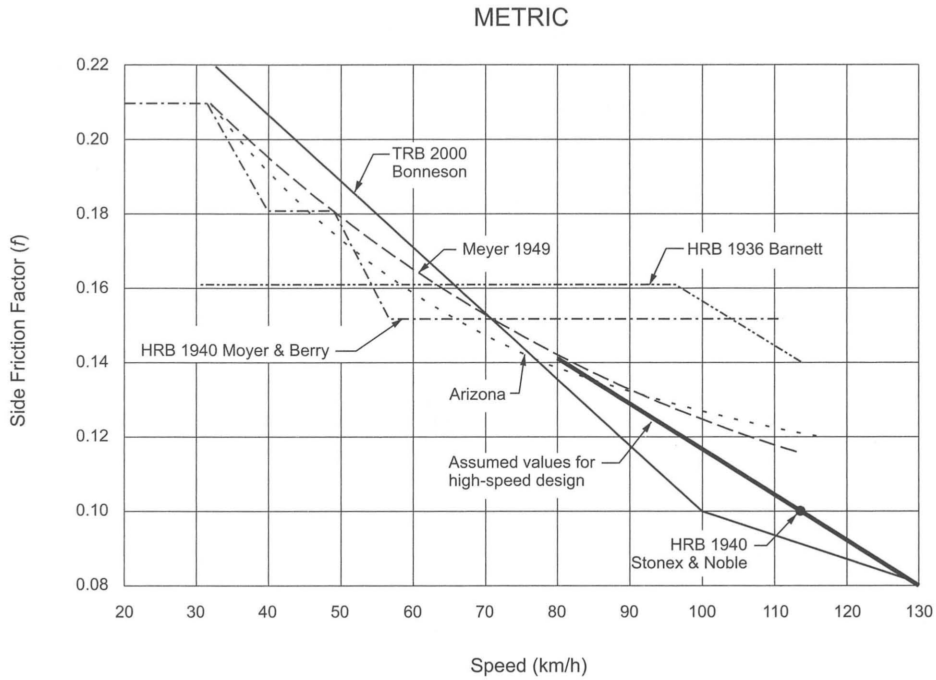
From other tests (*11*), a maximum side friction factor of 0.16 for speeds up to 100 km/h [60 mph] was recommended. For higher speeds, the incremental reduction of this factor was recommended. Speed studies on the Pennsylvania Turnpike (*60*) led to a conclusion that the side friction factor should not exceed 0.10 for design speeds of 110 km/h [70 mph] and higher. A recent study (*13*) re-examined previously published findings and analyzed new data collected at numerous horizontal curves. The side friction demand factors developed in that study are generally consistent with the side friction factors reported above.

An electronic accelerometer provides an alternative to the ball-bank indicator for use in determining advisory speeds for horizontal curves and ramps. An accelerometer is a gravity-sensitive electronic device that can measure the lateral forces and accelerations that drivers experience while traversing a highway curve (*20*).

It should be recognized that other factors influence driver speed choice under conditions of high friction demand. Swerving becomes perceptible, drift angle increases, and increased steering effort is needed to avoid involuntary lane line violations. Under these conditions, the cone of vision narrows and is accompanied by an increasing sense of concentration and intensity considered undesirable by most drivers. These factors are more apparent to a driver under open-road conditions.

Where practical, the maximum side friction factors used in design should be conservative for dry pavements and should provide an ample margin of safety against skidding on pavements that are wet as well as ice or snow covered and against vehicle rollover. The need to provide skid-resistant pavement surfacing for these conditions cannot be overemphasized because superimposed on the frictional demands resulting from roadway geometry are those that result from driving maneuvers such as braking, sudden lane changes, and minor changes in direction within a lane. In these short-term maneuvers, high friction demand can exist but the discomfort threshold may not be perceived in time for the driver to take corrective action.

Figure 3-4 summarizes the findings of the cited tests relating to side friction factors recommended for curve design. Although some variation in the test results is noted, all are in agreement that the side friction factor should be lower for high-speed design than for low-speed design. A recent study (*13*) reaffirmed the appropriateness of these side friction factors.



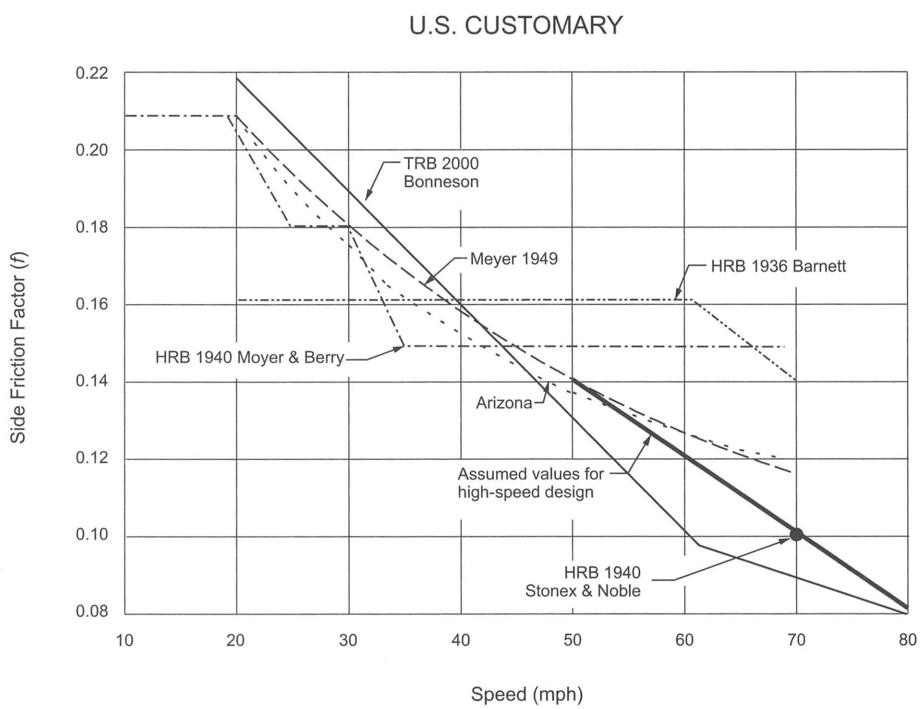


Figure 3-4. Side Friction Factors for High-Speed Streets and Highways

The maximum allowable side friction factors for low-speed streets and highways are shown in Figure 3-5. For travel on sharper curves, superelevation is needed. The curves are based on several studies (*14*, *16*, *23*) conducted to determine the side friction factor for low-speed intersection curves. A 95th percentile curve speed was used since it closely represents the 85th percentile tangent speed and provides a reasonable margin of safety against skidding (*13*). These curves also approximated the assumed values for low-speed urban design based on driver comfort. The curves provide an appropriate margin of safety against skidding and a cost-effective limitation on superelevation.

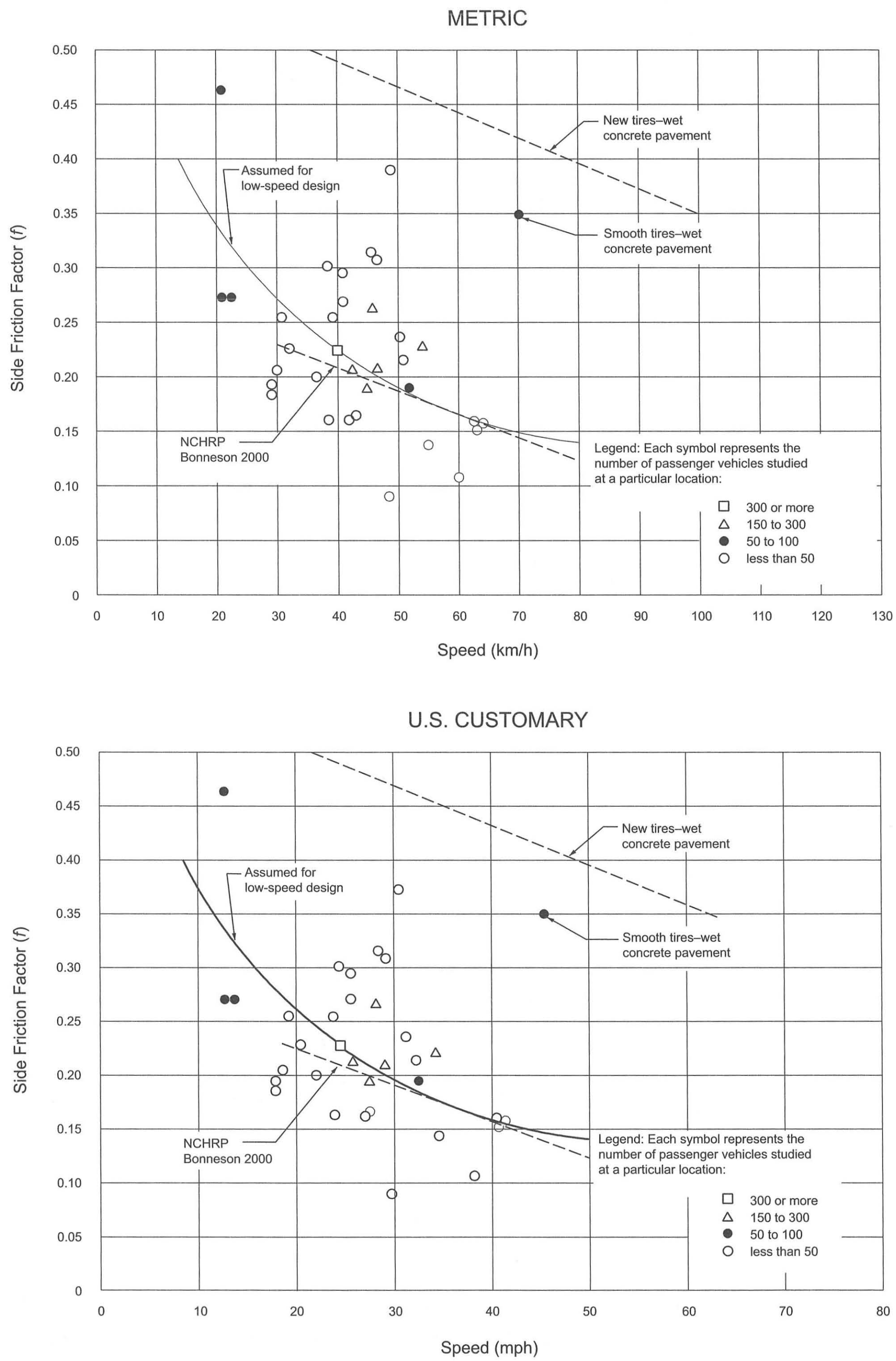


Figure 3-5. Side Friction Factors for Low-Speed Streets and Highways

The side friction factors vary with the design speed from 0.40 at 15 km/h [0.38 at 10 mph] to about 0.15 at 70 km/h [45 mph], with 70 km/h [45 mph] being the upper limit for low speed established in the design speed discussion in Section 2.3.6. Figure 3-6 should be referred to for the values of the side friction factor recommended for use in horizontal curve design.

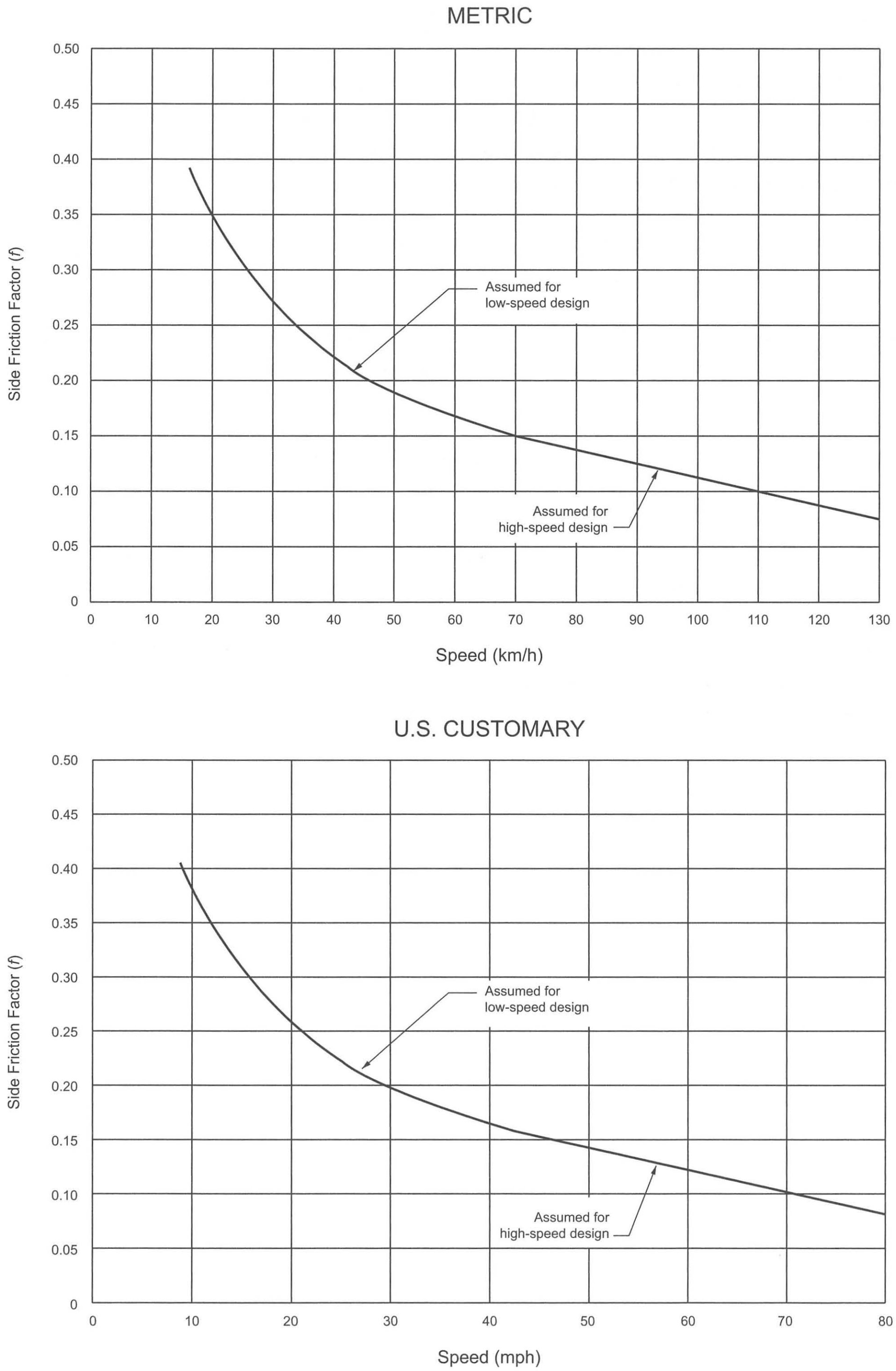


Figure 3-6. Side Friction Factors Assumed for Design

Distribution of *e* and *f* over a Range of Curves

For a given design speed there are five methods for sustaining lateral acceleration on curves by use of *e* or *f*, or both. These methods are discussed below, and the resulting relationships are illustrated in Figure 3-7:

- **Method 1**—Superelevation and side friction are directly proportional to the inverse of the radius (i.e., a straight-line relation exists between $1/R = 0$ and $1/R = 1/R_{min}$).
- **Method 2**—Side friction is such that a vehicle traveling at design speed has all lateral acceleration sustained by side friction on curves up to those designed for f_{max}. For sharper curves, *f* remains equal to f_{max} and superelevation is then used to sustain lateral acceleration until *e* reaches e_{max}. In this method, first *f* and then *e* are increased in inverse proportion to the radius of curvature.
- **Method 3**—Superelevation is such that a vehicle traveling at the design speed has all lateral acceleration sustained by superelevation on curves up to those designed for e_{max}.

  For sharper curves, *e* remains at e_{max} and side friction is then used to sustain lateral acceleration until *f* reaches f_{max}. In this method, first *e* and then *f* are increased in inverse proportion to the radius of curvature.
- **Method 4**—This method is the same as Method 3, except that it is based on average running speed instead of design speed.
- **Method 5**—Superelevation and side friction are in a curvilinear relation with the inverse of the radius of the curve, with values between those of Methods 1 and 3.

Figure 3-7A compares the relationship between superelevation and the inverse of the radius of the curve for these five methods. Figure 3-7B shows the corresponding value of side friction for a vehicle traveling at design speed, and Figure 3-7C for a vehicle traveling at the corresponding average running speed.

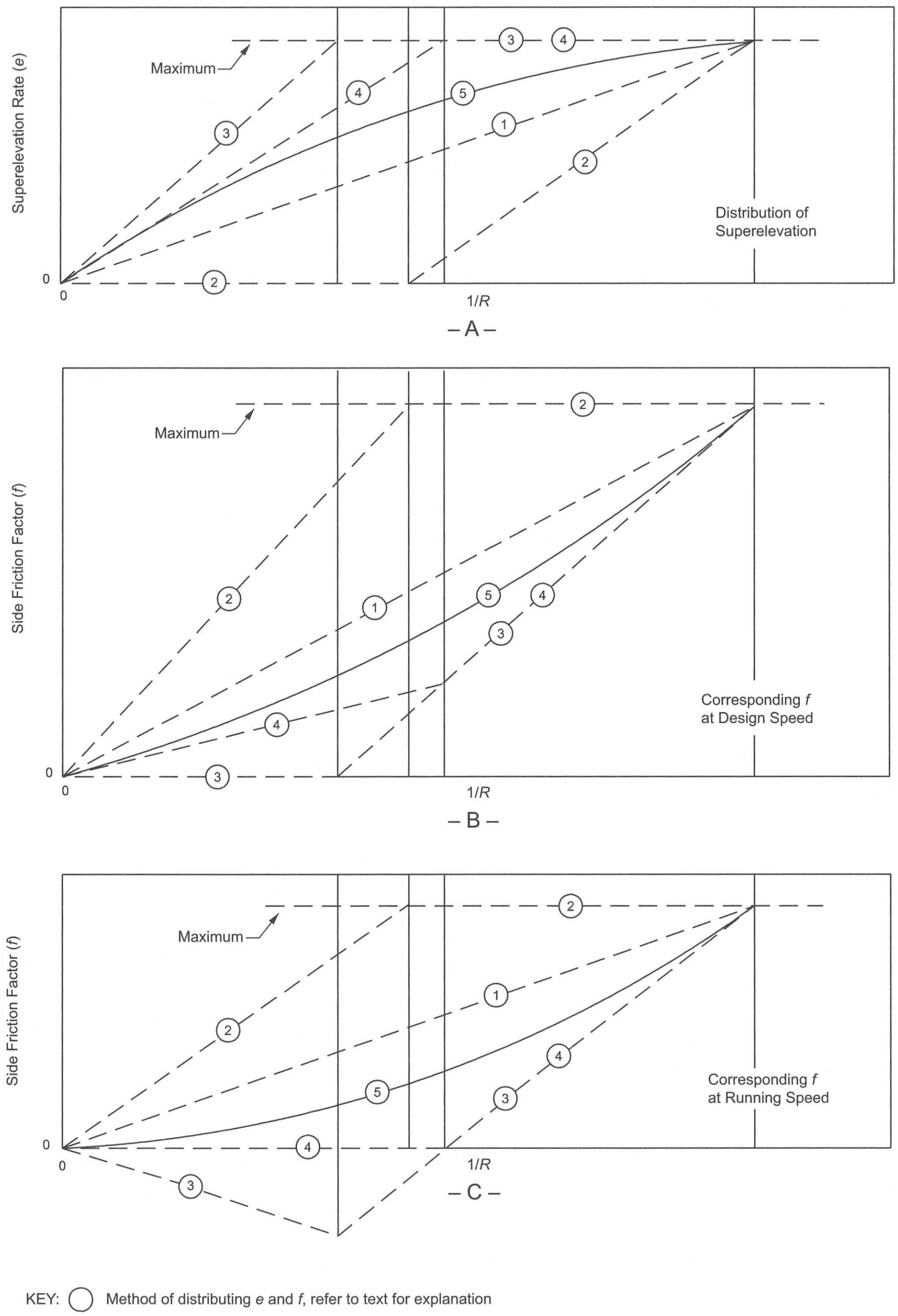


Figure 3-7. Methods of Distributing Superelevation and Side Friction

The straight-line relationship between superelevation and the inverse of the radius of the curve in Method 1 results in a similar relationship between side friction and the radius for vehicles traveling at either the design or average running speed. This method has considerable merit and logic in addition to its simplicity.

On any particular highway, the horizontal alignment consists of tangents and curves of varying radius greater than or equal to the minimum radius appropriate for the design speed (R_{min}). Application of superelevation in amounts directly proportional to the inverse of the radius would, for vehicles traveling at uniform speed, result in side friction factors with a straight-line variation from zero on tangents (ignoring cross slope) to the maximum side friction at the minimum radius. This method might appear to be an ideal means of distributing the side friction factor, but its appropriateness depends on travel at a constant speed by each vehicle in the traffic stream, regardless of whether travel is on a tangent, a curve of intermediate degree, or a curve with the minimum radius for that design speed. While uniform speed is the aim of most drivers, and can be obtained on well-designed highways when volumes are not heavy, there is a tendency for some drivers to travel faster on tangents and the flatter curves than on the sharper curves, particularly after being delayed by inability to pass slower moving vehicles. This tendency points to the desirability of providing superelevation rates for intermediate curves in excess of those that result from use of Method 1.

Method 2 uses side friction to sustain all lateral acceleration up to the curvature corresponding to the maximum side friction factor, and this maximum side friction factor is available on all sharper curves. In this method, superelevation is introduced only after the maximum side friction has been used. Therefore, no superelevation is needed on flatter curves that need less than maximum side friction for vehicles traveling at the design speed (see Curve 2 in Figure 3-7A). When superelevation is needed, it increases rapidly as curves with maximum side friction grow sharper. Because this method is completely dependent on available side friction, its use is generally limited to low-speed streets and highways. This method is particularly appropriate on low-speed urban streets where, because of various constraints, superelevation frequently cannot be provided.

In Method 3, which was practiced many years ago, superelevation to sustain all lateral acceleration for a vehicle traveling at the design speed is provided on all curves up to that needing maximum practical superelevation, and this maximum superelevation is provided on all sharper curves. Under this method, no side friction is provided on flat curves with less than maximum superelevation for vehicles traveling at the design speed, as shown by Curve 3 in Figure 3-7B, and the appropriate side friction increases rapidly as curves with maximum superelevation grow sharper. Further, as shown by Curve 3 in Figure 3-7C, for vehicles traveling at average running speed, this superelevation method results in negative friction for curves from very flat radii to about the middle of the range of curve radii; beyond this point, as curves become sharper, the side friction increases rapidly up to a maximum corresponding to the minimum radius of curvature. This marked difference in side friction for different curves is inconsistent and may result in erratic driving, either at the design or average running speed.

Method 4 is intended to overcome the deficiencies of Method 3 by using superelevation at speeds lower than the design speed. This method has been widely used with an average running speed for which all lateral acceleration is sustained by superelevation of curves flatter than that needing the maximum rate of superelevation. This average running speed was an approximation that, as presented in Table 3-6, varies from 78 to 100 [80 to 100] percent of design speed. Curve 4 in Figure 3-7A shows that in using this method the maximum superelevation is reached near the middle of the curvature range. Figure 3-7C shows that at average running speed no side friction is needed up to this curvature, and side friction increases rapidly and in direct proportion for sharper curves. This method has the same disadvantages as Method 3, but they apply to a smaller degree.

To accommodate overdriving that is likely to occur on flat to intermediate curves, it is desirable that the superelevation approximates that obtained by Method 4. Overdriving on such curves involves very little risk that a driver will lose control of the vehicle because superelevation sustains nearly all the lateral acceleration at the average running speed, and considerable side friction is available for greater speeds. On the other hand, Method 1, which avoids use of maximum superelevation for a substantial part of the range of curve radii, is also desirable. In Method 5, a curved line (Curve 5, as shown within the triangular working range between Curves 1 and 4 in Figure 3-7A) represents a superelevation and side friction distribution reasonably retaining the advantages of both Methods 1 and 4. Curve 5 has an asymmetrical parabolic form and represents a practical distribution for superelevation over the range of curvature.

Table 3-6. Average Running Speeds

Metric		U.S. Customary	
Design Speed (km/h)	**Average Running Speed (km/h)**	**Design Speed (mph)**	**Average Running Speed (mph)**
20	20	15	15
30	30	20	20
40	40	25	24
50	47	30	28
60	55	35	32
70	63	40	36
80	70	45	40
90	77	50	44
100	85	55	48
110	91	60	52
120	98	65	55
130	102	70	58
		75	61
		80	64

3.3.3 Design Considerations

Superelevation rates that are applicable over the range of curvature for each design speed have been determined for use in highway design. One extreme of this range is the maximum superelevation rate established by practical considerations and used to determine the maximum curvature for each design speed. The maximum superelevation may be different for different highway conditions. At the other extreme, no superelevation is needed for tangent highways or highways with extremely long-radius curves. For curvature between these extremes and for a given design speed, the superelevation should be chosen in such a manner that there is a logical relation between the side friction factor and the applied superelevation rate.

Normal Cross Slope

The minimum rate of cross slope applicable to the traveled way is determined by drainage needs. Consistent with the type of highway and amount of rainfall, snow, and ice, the usually accepted minimum values for cross slope range from 1.5 percent to 2.0 percent (for further information, see Section 4.2.2 on "Cross Slope"). For discussion purposes, a value of 2.0 percent is used in this discussion as a single value representative of the cross slope for paved, uncurbed pavements. Steeper cross slopes are generally needed where curbs are used to minimize ponding of water on the outside through lane.

The shape or form of the normal cross slope varies. Some states and many municipalities use a curved traveled way cross section for two-lane roadways, usually parabolic in form. Others employ a straight-line section for each lane.

Maximum Superelevation Rates for Streets and Highways

The maximum rates of superelevation used on highways are controlled by four factors: climate conditions (i.e., frequency and amount of snow and ice); terrain conditions (i.e., flat, rolling, or mountainous); type of area (i.e., rural or urban); and frequency of very slow-moving vehicles whose operation might be affected by high superelevation rates. Consideration of these factors jointly leads to the conclusion that no single maximum superelevation rate is universally applicable. However, using only one maximum superelevation rate within a region of similar climate and land use is desirable, as such a practice promotes design consistency.

Design consistency represents the uniformity of the highway alignment and its associated design element dimensions. This uniformity allows drivers to improve their perception-reaction skills by developing expectancies. Design elements that are not uniform for similar types of roadways may be counter to a driver's expectancy and result in an increase in driver workload. Logically, there is an inherent relationship between design consistency, driver workload, and crash frequency, with "consistent" designs being associated with lower workloads and lower crash frequencies.

The highest superelevation rate for highways in common use is 10 percent, although 12 percent is used in some cases. Superelevation rates above 8 percent are only used in areas without snow and ice. Although higher superelevation rates offer an advantage to those drivers traveling at high speeds, current practice considers that rates in excess of 12 percent are beyond practical limits. This practice recognizes the combined effects of construction processes, maintenance difficulties, and operation of vehicles at low speeds.

Thus, a superelevation rate of 12 percent appears to represent a practical maximum value where snow and ice do not exist. A superelevation rate of 12 percent may be used on low-volume gravel-surfaced roads to facilitate cross drainage; however, superelevation rates of this magnitude can cause higher speeds, which are conducive to rutting and displacement of gravel. Generally, 8 percent is recognized as a reasonable maximum value for superelevation rate.

Where snow and ice are factors, tests and experience show that a superelevation rate of about 8 percent is a logical maximum to minimize vehicles sliding across a highway when stopping or attempting to start slowly from a stopped position. One series of tests (*46*) found coefficients of friction for ice ranging from 0.050 to 0.200, depending on the condition of the ice (i.e., wet, dry, clean, smooth, or rough). Tests on loose or packed snow show coefficients of friction ranging from 0.200 to 0.400. Other tests (*27*) have corroborated these values. The lower extreme of this range of coefficients of friction probably occurs only under thin film "quick freeze" conditions at a temperature of about –1°C [30°F] in the presence of water on the pavement. Similar low friction values may occur with thin layers of mud on the pavement surface, with oil or flushed spots, and with high speeds and a sufficient depth of water on the pavement surface to permit hydroplaning. For these reasons, some highway agencies have adopted a maximum superelevation rate of 8 percent. Such agencies believe that 8 percent represents a logical maximum superelevation rate, regardless of snow or ice conditions. Such a limit tends to reduce the likelihood that slow drivers will experience negative side friction, which can result in excessive steering effort and erratic operation.

Where traffic congestion or extensive marginal development acts to restrict top speeds, it is common practice to utilize a lower maximum rate of superelevation, usually 4 to 6 percent. Similarly, either a low maximum rate of superelevation or no superelevation is employed within important intersection areas or where there is a tendency to drive slowly because of turning and crossing movements, warning devices, and signals. In these areas it is difficult to warp crossing pavements for drainage without providing negative superelevation for some turning movements.

In summary, it is recommended that (1) several rates, rather than a single rate, of maximum superelevation should be recognized in establishing design controls for highway curves, (2) a rate of 12 percent should not be exceeded, (3) a rate of 4 or 6 percent is applicable for urban design in areas with few constraints, and (4) superelevation may be omitted on low-speed urban streets where severe constraints are present. To account for a wide range of agency practice, five maximum superelevation rates—4, 6, 8, 10, and 12 percent—are presented in this chapter.

Minimum Radius

The minimum radius is a limiting value of curvature for a given design speed and is determined from the maximum rate of superelevation and the maximum side friction factor selected for design (limiting value of f). Use of sharper curvature for that design speed would call for superelevation beyond the limit considered practical or for operation with tire friction and lateral acceleration beyond what is considered comfortable by many drivers, or both. The minimum radius of curvature is based on a threshold of driver comfort that is sufficient to provide a margin of safety against skidding and vehicle rollover. The minimum radius of curvature is also an important control value for determining superelevation rates for flatter curves.

The minimum radius of curvature, R_{min}, can be calculated directly from the simplified curve equation (see Equation 3-7) introduced previously in Section 3.3.2 under "Side Friction Factor." This equation can be recast to determine R_{min} as follows:

Metric	U.S. Customary	
$R_{min} = \dfrac{V^2}{127(0.01e_{max} + f_{max})}$	$R_{min} = \dfrac{V^2}{15(0.01e_{max} + f_{max})}$	(3-8)

Based on the maximum allowable side friction factors from Figure 3-6, Table 3-7 gives the minimum radius for each of the five maximum superelevation rates calculated using Equation 3-8.

For curve layout purposes, the radius is measured to the horizontal control line, which is often along the centerline of the alignment. However, the horizontal curve equations use a curve radius measured to a vehicle's center of gravity, which is approximately the center of the innermost travel lane. The equations do not consider the width of the roadway or the location of the horizontal control line. For consistency with the radius defined for turning roadways and to consider the motorist operating within the innermost travel lane, the radius used to design horizontal curves should be measured to the inside edge of the innermost travel lane, particularly for wide roadways with sharp horizontal curvature. For two-lane roadways, the difference between the roadway centerline and the center of gravity used in the horizontal curve equations is minor. Therefore, the curve radius for a two-lane roadway may be measured to the centerline of the roadway.

Table 3-7. Minimum Radius Using Limiting Values of *e* and *f*

Metric						U.S. Customary					
Design Speed (km/h)	Maxi-mum *e* (%)	Maxi-mum *f*	Total (*e*/100 + *f*)	Calcu-lated Radius (m)	Rounded Radius (m)	Design Speed (mph)	Maxi-mum *e* (%)	Maxi-mum *f*	Total (*e*/100 + *f*)	Calcu-lated Radius (ft)	Rounded Radius (ft)
15	4.0	0.40	0.44	4.0	4	10	4.0	0.38	0.42	15.9	16
20	4.0	0.35	0.39	8.1	8	15	4.0	0.32	0.36	41.7	42
30	4.0	0.28	0.32	22.1	22	20	4.0	0.27	0.31	86.0	86
40	4.0	0.23	0.27	46.7	47	25	4.0	0.23	0.27	154.3	154
50	4.0	0.19	0.23	85.6	86	30	4.0	0.20	0.24	250.0	250
60	4.0	0.17	0.21	135.0	135	35	4.0	0.18	0.22	371.2	371
70	4.0	0.15	0.19	203.1	203	40	4.0	0.16	0.20	533.3	533
80	4.0	0.14	0.18	280.0	280	45	4.0	0.15	0.19	710.5	711
90	4.0	0.13	0.17	375.2	375	50	4.0	0.14	0.18	925.9	926
100	4.0	0.12	0.16	492.1	492	55	4.0	0.13	0.17	1186.3	1190
						60	4.0	0.12	0.16	1500.0	1500
15	6.0	0.40	0.46	3.9	4	10	6.0	0.38	0.44	15.2	15
20	6.0	0.35	0.41	7.7	8	15	6.0	0.32	0.38	39.5	39
30	6.0	0.28	0.34	20.8	21	20	6.0	0.27	0.33	80.8	81
40	6.0	0.23	0.29	43.4	43	25	6.0	0.23	0.29	143.7	144
50	6.0	0.19	0.25	78.7	79	30	6.0	0.20	0.26	230.8	231
60	6.0	0.17	0.23	123.2	123	35	6.0	0.18	0.24	340.3	340
70	6.0	0.15	0.21	183.7	184	40	6.0	0.16	0.22	484.8	485
80	6.0	0.14	0.20	252.0	252	45	6.0	0.15	0.21	642.9	643
90	6.0	0.13	0.19	335.7	336	50	6.0	0.14	0.20	833.3	833
100	6.0	0.12	0.18	437.4	437	55	6.0	0.13	0.19	1061.4	1060
110	6.0	0.11	0.17	560.4	560	60	6.0	0.12	0.18	1333.3	1330
120	6.0	0.09	0.15	755.9	756	65	6.0	0.11	0.17	1656.9	1660
130	6.0	0.08	0.14	950.5	951	70	6.0	0.10	0.16	2041.7	2040
						75	6.0	0.09	0.15	2500.0	2500
						80	6.0	0.08	0.14	3047.6	3050
15	8.0	0.40	0.48	3.7	4	10	8.0	0.38	0.46	14.5	14
20	8.0	0.35	0.43	7.3	7	15	8.0	0.32	0.40	37.5	38
30	8.0	0.28	0.36	19.7	20	20	8.0	0.27	0.35	76.2	76
40	8.0	0.23	0.31	40.6	41	25	8.0	0.23	0.31	134.4	134
50	8.0	0.19	0.27	72.9	73	30	8.0	0.20	0.28	214.3	214
60	8.0	0.17	0.25	113.4	113	35	8.0	0.18	0.26	314.1	314
70	8.0	0.15	0.23	167.8	168	40	8.0	0.16	0.24	444.4	444
80	8.0	0.14	0.22	229.1	229	45	8.0	0.15	0.23	587.0	587
90	8.0	0.13	0.21	303.7	304	50	8.0	0.14	0.22	757.6	758
100	8.0	0.12	0.20	393.7	394	55	8.0	0.13	0.21	960.3	960
110	8.0	0.11	0.19	501.5	501	60	8.0	0.12	0.20	1200.0	1200
120	8.0	0.09	0.17	667.0	667	65	8.0	0.11	0.19	1482.5	1480
130	8.0	0.08	0.16	831.7	832	70	8.0	0.10	0.18	1814.8	1810
						75	8.0	0.09	0.17	2205.9	2210
						80	8.0	0.08	0.16	2666.7	2670
15	10.0	0.40	0.50	3.5	4	10	10.0	0.38	0.48	13.9	14
20	10.0	0.35	0.45	7.0	7	15	10.0	0.32	0.42	35.7	36
30	10.0	0.28	0.38	18.6	19	20	10.0	0.27	0.37	72.1	72
40	10.0	0.23	0.33	38.2	38	25	10.0	0.23	0.33	126.3	126
50	10.0	0.19	0.29	67.9	68	30	10.0	0.20	0.30	200.0	200
60	10.0	0.17	0.27	105.0	105	35	10.0	0.18	0.28	291.7	292
70	10.0	0.15	0.25	154.3	154	40	10.0	0.16	0.26	410.3	410
80	10.0	0.14	0.24	210.0	210	45	10.0	0.15	0.25	540.0	540
90	10.0	0.13	0.23	277.3	277	50	10.0	0.14	0.24	694.4	694
100	10.0	0.12	0.22	357.9	358	55	10.0	0.13	0.23	876.8	877
110	10.0	0.11	0.21	453.7	454	60	10.0	0.12	0.22	1090.9	1090
120	10.0	0.09	0.19	596.8	597	65	10.0	0.11	0.21	1341.3	1340
130	10.0	0.08	0.18	739.3	739	70	10.0	0.10	0.20	1633.3	1630
						75	10.0	0.09	0.19	1973.7	1970
						80	10.0	0.08	0.18	2370.4	2370
15	12.0	0.40	0.52	3.4	3	10	12.0	0.38	0.50	13.3	13
20	12.0	0.35	0.47	6.7	7	15	12.0	0.32	0.44	34.1	34
30	12.0	0.28	0.40	17.7	18	20	12.0	0.27	0.39	68.4	68
40	12.0	0.23	0.35	36.0	36	25	12.0	0.23	0.35	119.0	119
50	12.0	0.19	0.31	63.5	64	30	12.0	0.20	0.32	187.5	188
60	12.0	0.17	0.29	97.7	98	35	12.0	0.18	0.30	272.2	272
70	12.0	0.15	0.27	142.9	143	40	12.0	0.16	0.28	381.0	381
80	12.0	0.14	0.26	193.8	194	45	12.0	0.15	0.27	500.0	500
90	12.0	0.13	0.25	255.1	255	50	12.0	0.14	0.26	641.0	641
100	12.0	0.12	0.24	328.1	328	55	12.0	0.13	0.25	806.7	807
110	12.0	0.11	0.23	414.2	414	60	12.0	0.12	0.24	1000.0	1000
120	12.0	0.09	0.21	539.9	540	65	12.0	0.11	0.23	1224.6	1220
130	12.0	0.08	0.20	665.4	665	70	12.0	0.10	0.22	1484.8	1480
						75	12.0	0.09	0.21	1785.7	1790
						80	12.0	0.08	0.20	2133.3	2130

Note: In recognition of safety considerations, use of e_{max} = 4.0% should be limited to urban conditions.

Effects of Grades

On long or fairly steep grades, drivers tend to travel faster in the downgrade than in the upgrade direction. Additionally, research (*13*) has shown that the side friction demand is greater on both downgrades (due to braking forces) and steep upgrades (due to the tractive forces). Some adjustment in superelevation rates should be considered for grades steeper than 5 percent. This adjustment is particularly important on facilities with high truck volumes and on low-speed facilities with intermediate curves using high levels of side friction demand.

In the case of a divided highway with each roadway independently superelevated, or on a one-way ramp, such an adjustment can be readily made. In the simplest practical form, values from Tables 3-8 to 3-12, presented in Section 3.3.5, can be used directly by assuming a slightly higher design speed for the downgrade. Since vehicles tend to slow on steep upgrades, the superelevation adjustment can be made without reducing the design speed for the upgrade. The appropriate variation in speed depends on the particular conditions, especially the rate and length of grade and the magnitude of the curve radius compared to other curves on the approach highway section.

On two-lane and multilane undivided roadways, the adjustment for grade can be made by assuming a slightly higher design speed for the downgrade and applying it to the whole traveled way (both upgrade and downgrade sides). The added superelevation for the upgrade can help counter the loss of available side friction due to tractive forces. On long upgrades, the additional superelevation may cause negative side friction for slow moving vehicles (such as large trucks). This effect is mitigated by the slow speed of the vehicle, allowing time to counter steer, and the increased experience and training for truck drivers.

3.3.4 Design for Rural Highways, Urban Freeways, and High-Speed Urban Streets

On rural highways, on urban freeways, and on urban streets where speed is relatively high and relatively uniform, horizontal curves are generally superelevated and successive curves are generally balanced to provide a smooth-riding transition from one curve to the next. A balanced design for a series of curves of varying radii is provided by the appropriate distribution of e and f values, as discussed above, to select an appropriate superelevation rate in the range from the normal cross slope to maximum superelevation.

Side Friction Factors

Figure 3-6 shows the recommended side friction factors for rural highways, urban freeways, and high-speed urban streets and highways as a solid line. They provide a reasonable margin of safety for the various speeds.

The maximum side friction factors vary directly with design speed from 0.14 at 80 km/h [50 mph] to 0.08 at 130 km/h [80 mph]. The research report *Side Friction for Superelevation on Horizontal Curves* (*42*) confirms the appropriateness of these design values.

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 rural highways, urban freeways, and high-speed urban streets. Use of Method 5 is discussed in the following text and figures.

Procedure for Development of Method 5 Superelevation Distribution

The side friction factors shown as the solid line on Figure 3-6 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 3-8). The finalized *e* distribution curves resulting from this approach, based on Method 5 and used below, are shown in Figures 3-9 to 3-13.

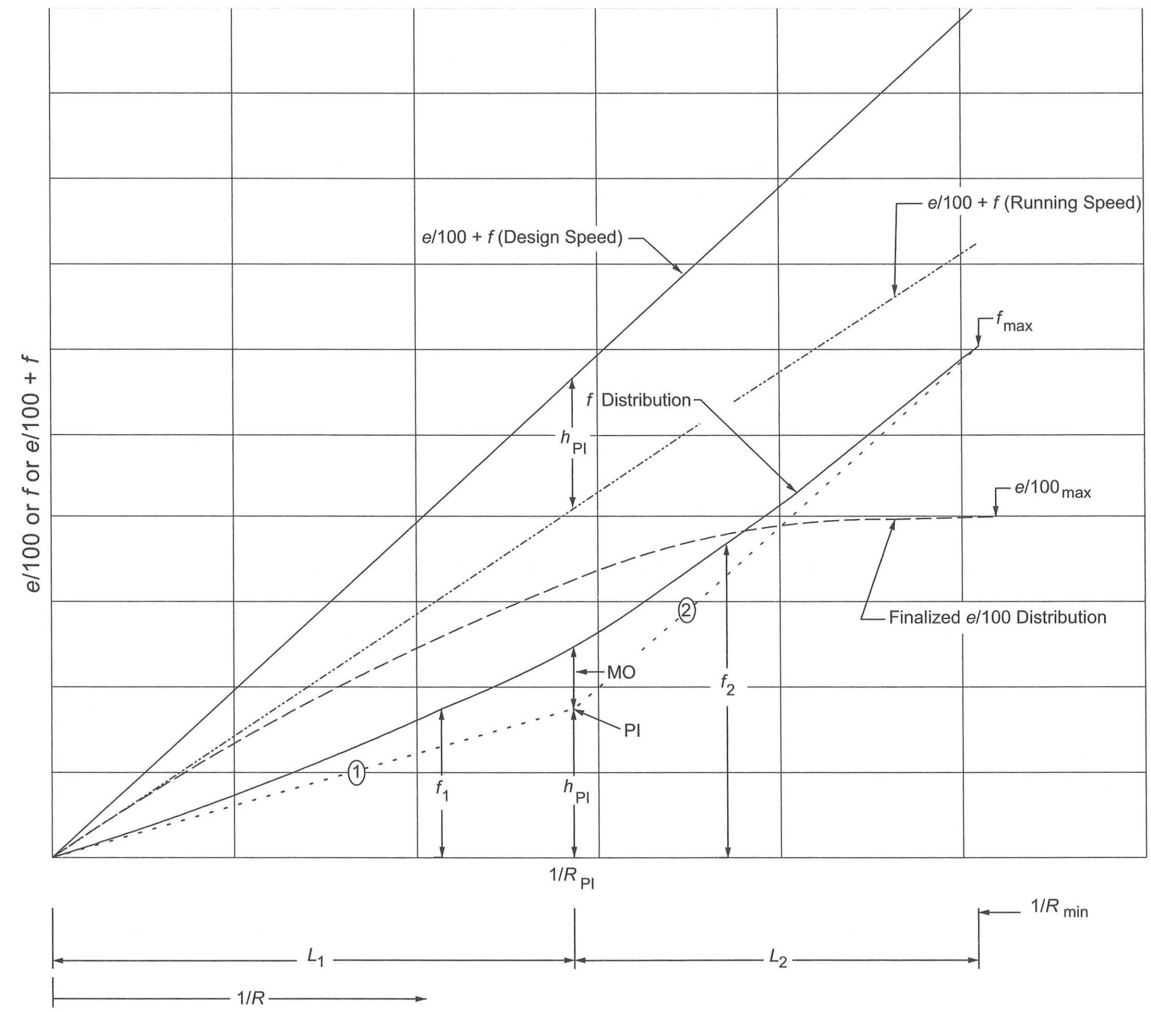


Figure 3-8. Method 5 Procedure for Development of the Superelevation Distribution

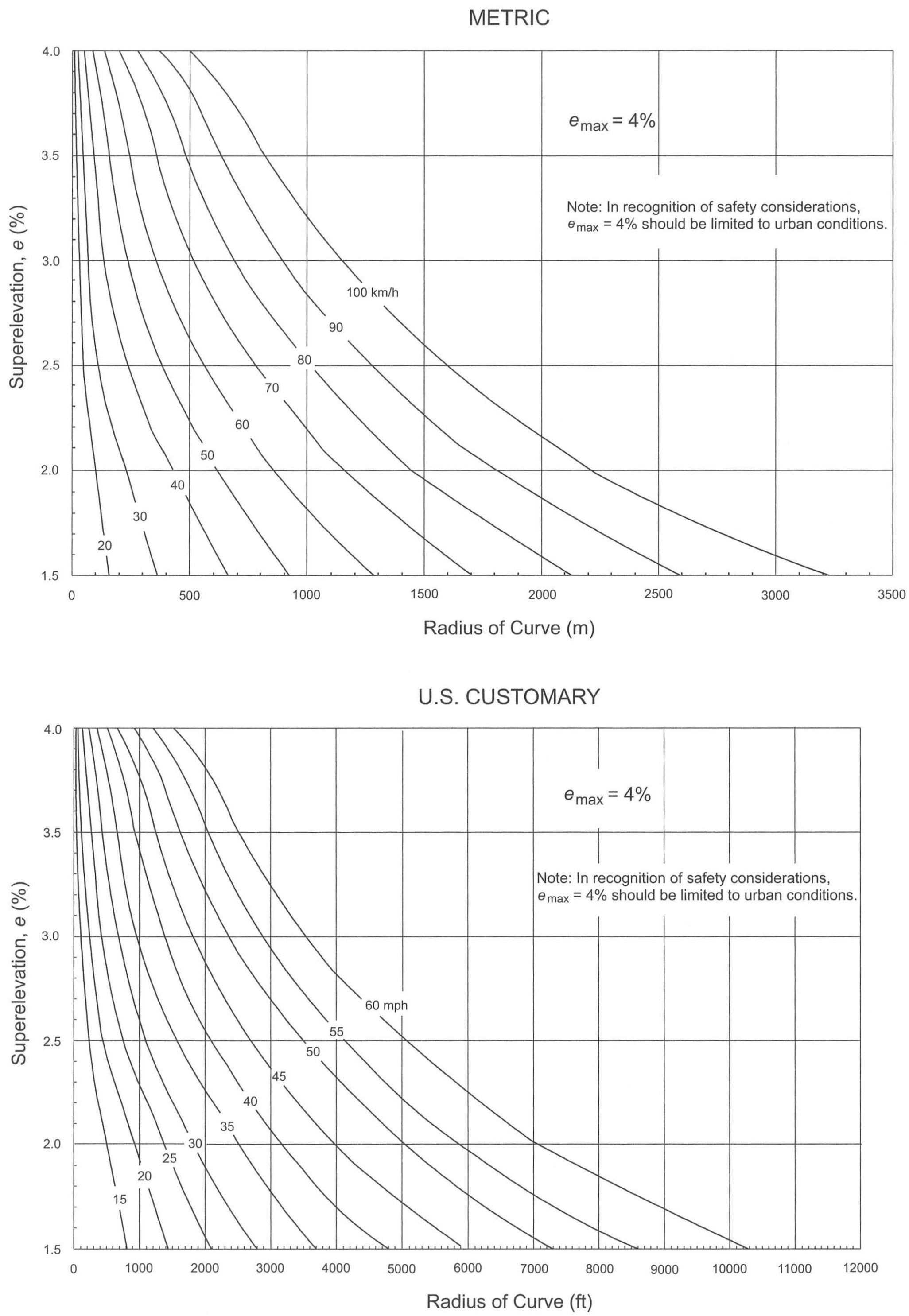


Figure 3-9. Design Superelevation Rates for Maximum Superelevation Rate of 4 Percent

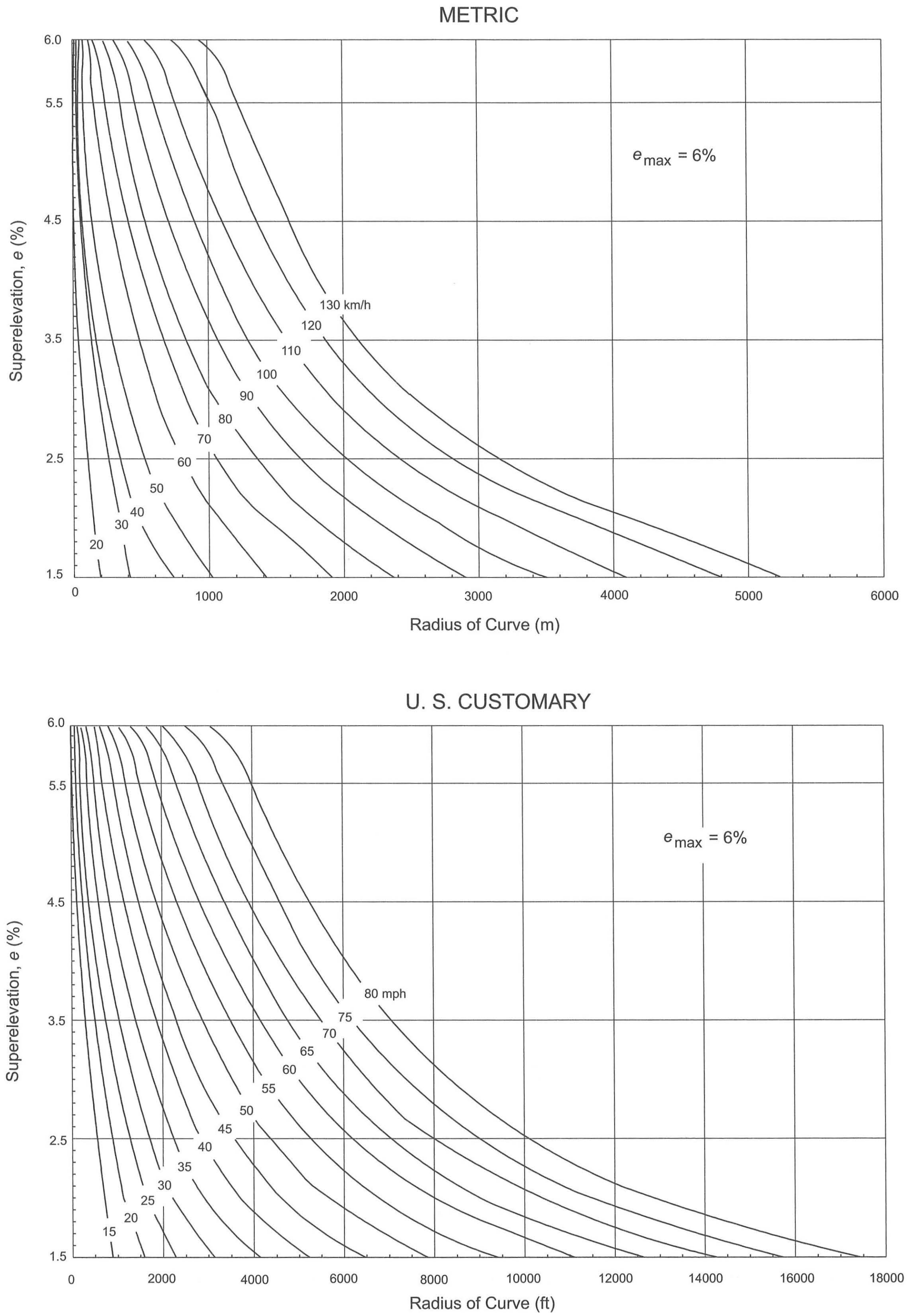
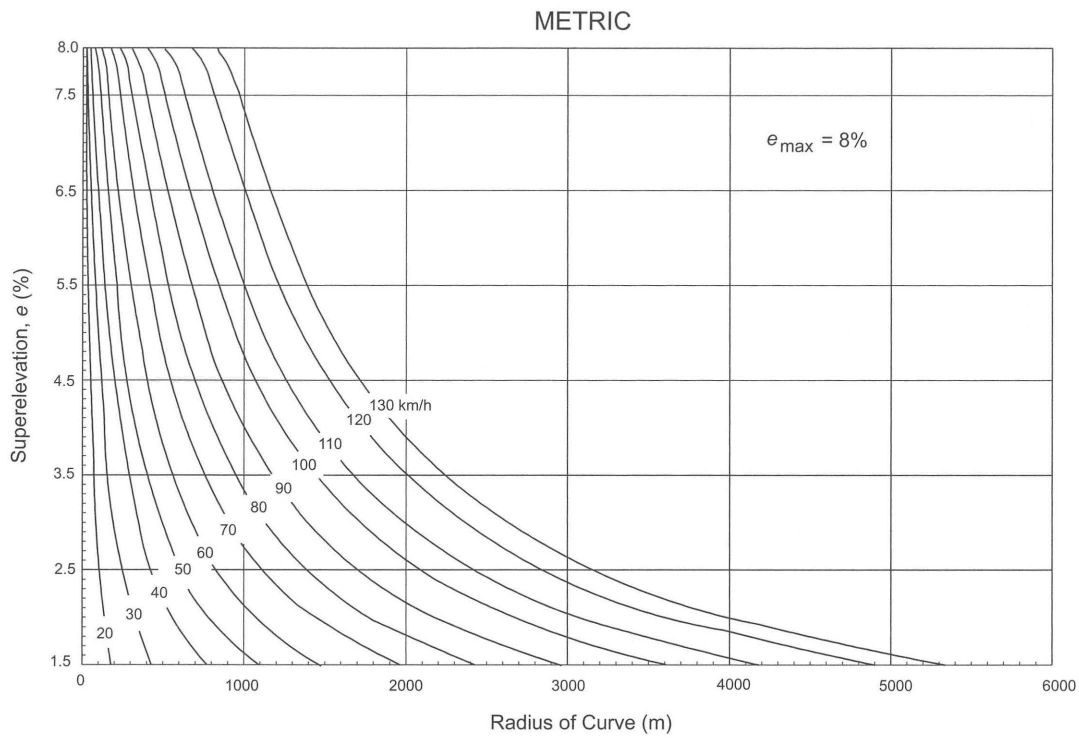


Figure 3-10. Design Superelevation Rates for Maximum Superelevation Rate of 6 Percent



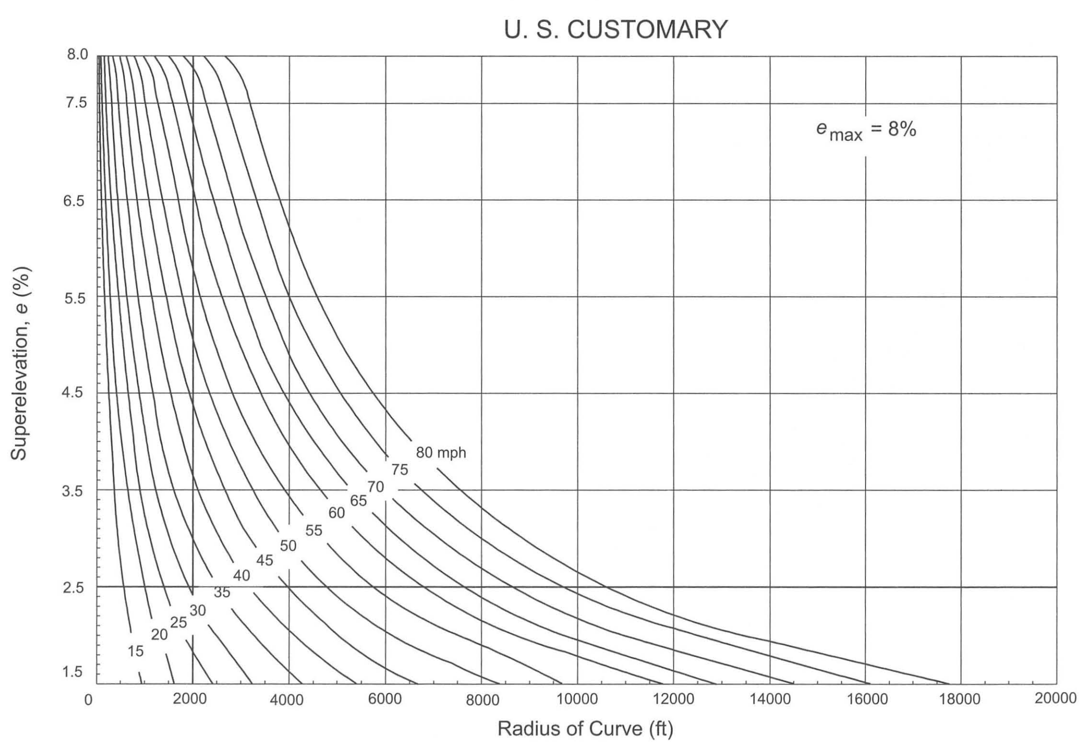
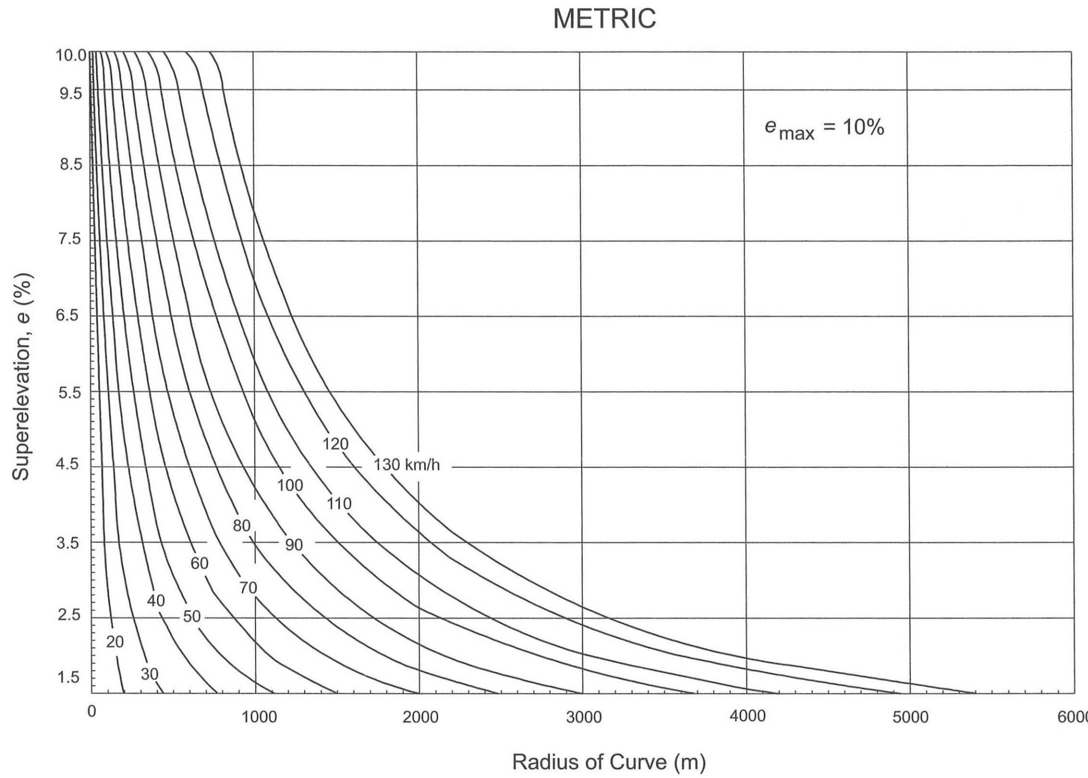


Figure 3-11. Design Superelevation Rates for Maximum Superelevation Rate of 8 Percent



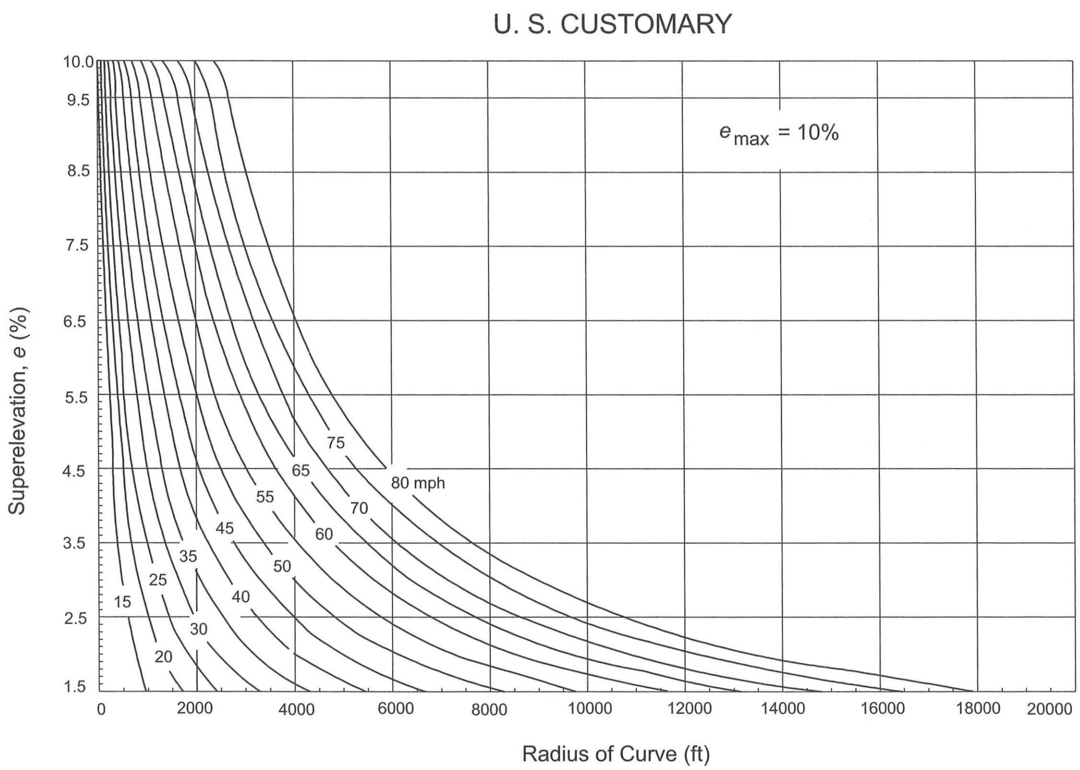
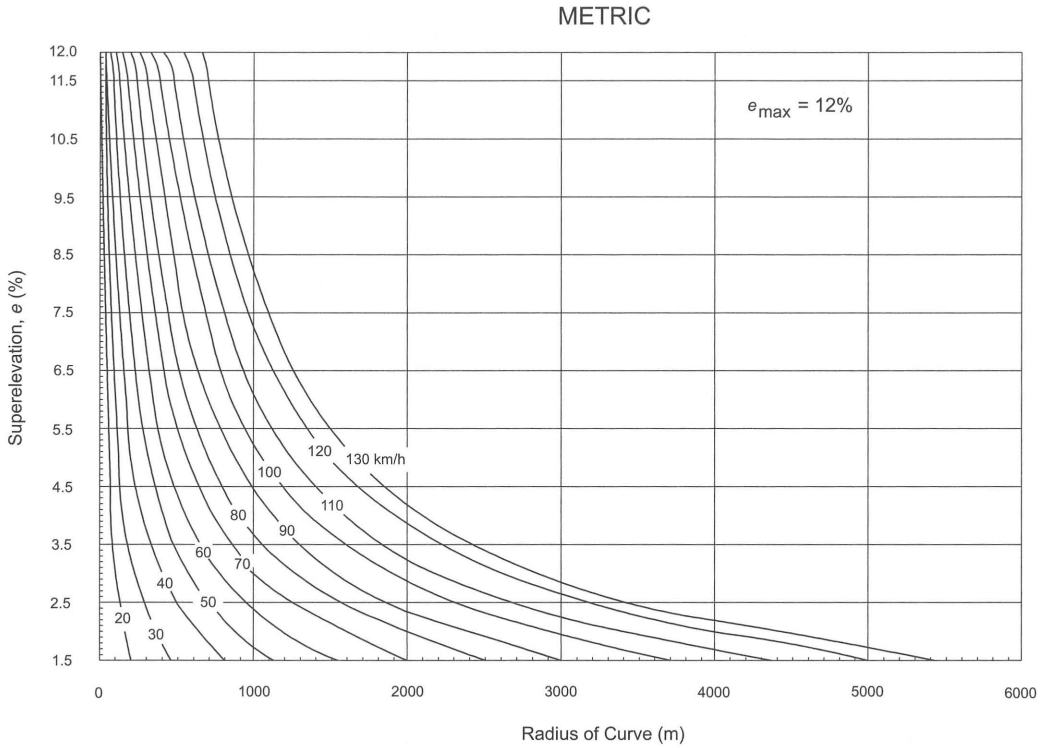


Figure 3-12. Design Superelevation Rates for Maximum Superelevation Rate of 10 Percent



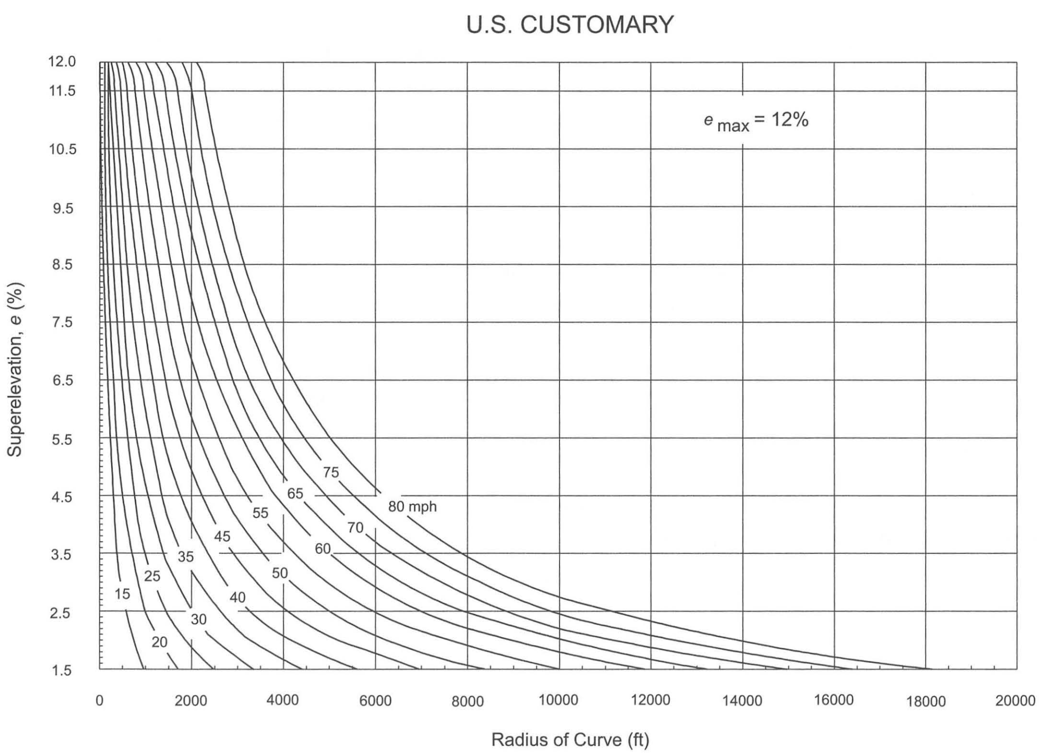


Figure 3-13. Design Superelevation Rates for Maximum Superelevation Rate of 12 Percent

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

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

Metric	U.S. Customary	
where S_1 = slope of leg 1 and	where S_1 = slope of leg 1 and	
$$S_2 = \frac{f_{\max} - h_{PI}}{\frac{1}{R_{\min}} - \frac{1}{R_{PI}}}$$	$$S_2 = \frac{f_{max} - h_{PI}}{5729.58\left(\frac{1}{R_{min}} - \frac{1}{R_{PI}}\right)}$$	(3-14)
where S_2 = slope of leg 2.	where S_2 = slope of leg 2.	
The equation for the middle ordinate (MO) of an unsymmetrical vertical curve is the following:	The equation for the middle ordinate (MO) of an unsymmetrical vertical curve is the following:	
$$MO = \frac{L_1 L_2 (S_2 - S_1)}{2(L_1 + L_2)}$$	$$MO = \frac{L_1 L_2 (S_2 - S_1)}{2(L_1 + L_2)}$$	(3-15)
where: $L_1 = 1/R_{PI}$ and $L_2 = 1/R_{min} - 1/R_{PI}$. It follows that:	where: $L_1 = 5729.58/R_{PI}$ and $L_2 = 5729.58(1/R_{min} - 1/R_{PI})$. It follows that:	
$$MO = \frac{1}{R_{PI}}\left(\frac{1}{R_{\min}} - \frac{1}{R_{PI}}\right)\left(\frac{S_2 - S_1}{2}\right)R_{\min}$$	$$MO = \frac{5729.58}{R_{PI}}\left(\frac{1}{R_{\min}} - \frac{1}{R_{PI}}\right)\left(\frac{S_2 - S_1}{2}\right)(R_{\min})$$	(3-16)
where MO = middle ordinate of the f distribution curve, and	where MO = middle ordinate of the f distribution curve, and	
$$(0.01e + f)_D = \frac{(0.01e_{\max} + f_{\max})R_{\min}}{R}$$	$$(0.01e + f)_D = \frac{(0.01e_{\max} + f_{\max})R_{\min}}{R}$$	(3-17)
in which R = radius at any point.	in which R = radius at any point.	
Use the general vertical curve equation:	Use the general vertical curve equation:	
$$\frac{Y}{MO} = \left(\frac{x}{L}\right)^2$$	$$\frac{Y}{MO} = \left(\frac{x}{L}\right)^2$$	(3-18)
with $1/R$ measured from the vertical axis. With $1/R \le 1/R_{PI}$,	with $1/R$ measured from the vertical axis. With $1/R \le 1/R_{PI}$,	
$$f_1 = MO\left(\frac{R_{PI}}{R}\right)^2 + \frac{S_1}{R}$$	$$f_1 = MO\left(\frac{R_{PI}}{R}\right)^2 + \frac{5729.58(S_1)}{R}$$	(3-19)
where: f_1 = f distribution at any point $1/R \le 1/R_{PI}$; and	where: f_1 = f distribution at any point $1/R \le 1/R_{PI}$; and	

Metric	U.S. Customary	
$0.01e_1 = (0.01e + f)_D - f_1$	$0.01e_1 = (0.01e + f)_D - f_1$	(3-20)
where: $0.01e_1 = 0.01e$ distribution at any point $1/R \leq 1/R_{PI}$.	where: $0.01e_1 = 0.01e$ distribution at any point $1/R \leq 1/R_{PI}$.	
For $1/R > 1/R_{PI}$,	For $1/R > 1/R_{PI}$,	
$f_2 = MO\left(\frac{\frac{1}{R_{min}} - \frac{1}{R}}{\frac{1}{R_{min}} - \frac{1}{R_{PI}}}\right)^2 + h_{PI} + S_2\left(\frac{1}{R} - \frac{1}{R_{PI}}\right)$	$f = MO\left(\frac{\frac{1}{R_{min}} - \frac{1}{R}}{\frac{1}{R_{min}} - \frac{1}{R_{PI}}}\right)^2 + h_{PI} + 5729.58(S_2)\left(\frac{1}{R} - \frac{1}{R_{PI}}\right)$	(3-21)
where: $f_2 = f$ distribution at any point $1/R > 1/R_{PI}$; and	where: $f_2 = f$ distribution at any point $1/R > 1/R_{PI}$; and	
$0.01e_2 = (0.01e + f)_D - f_2$	$0.01e_2 = (0.01e + f)_D - f_2$	(3-22)
where: $0.01e_2 = 0.01e$ distribution at any point $1/R > 1/R_{PI}$.	where: $0.01e_2 = 0.01e$ distribution at any point $1/R > 1/R_{PI}$.	

Figure 3-8 is a typical layout illustrating the Method 5 procedure for development of the finalized e distribution. The figure depicts how the f value is determined for $1/R$ and then subtracted from the value of $(e/100 + f)$ to determine $e/100$.

An example of the procedure to calculate e for a design speed of 80 km/h [50 mph] and an e_{max} of 8 percent is shown below:

Example

Metric	U.S. Customary
Determine e given: $V_D = 80$ km/h $e_{max} = 8$ percent	Determine e given: $V_D = 50$ mph $e_{max} = 8$ percent
From Table 3-6: $V_R = 70$ km/h From Table 3-7: $f = 0.14$ (maximum allowable side friction factor)	From Table 3-6: $V_R = 44$ mph From Table 3-7: $f = 0.14$ (maximum allowable side friction factor)
Using the appropriate equations yields: $R_{min} = 229.1$, $R_{PI} = 482.3$, and $h_{PI} = 0.02449$	Using the appropriate equations yields: $R_{min} = 757.6$, $R_{PI} = 1613$, and $h_P = 0.02331$
$S_1 = 11.81$ and $S_2 = 50.41$ $L_1 = 0.002073$ and $L_2 = 0.002292$ The middle ordinate (MO) is 0.02101.	$S_1 = 0.006562$ and $S_2 = 0.02910$ $L_1 = 3.551$ and $L_2 = 4.012$ The middle ordinate (MO) is 0.02122.

Example	
Metric	**U.S. Customary**
The e distribution value for any radius is found by taking the $(0.01e+f)_D$ value minus the f_1 or f_2 value (refer to Figure 3-8). Thus, the e distribution value for an $R = R_{PI}$ would be $(0.01e+f)$ $D = VD^2/127R = 0.1045$ minus an $f_1 = 0.0455$, which results in 0.05899. This value, multiplied by 100 (to convert to percent) and rounded up to the nearest 0.2 of a percent, corresponds to an e value of 6.0 percent. This e value can be found for $R = 482.3$ m at the 80 km/h design speed in Table 3-10.	The e distribution value for any radius is found by taking the $(0.01e+f)_D$ value minus the f_1 or f_2 value (refer to Figure 3-8). Thus, the e distribution value for an $R = R_{PI}$ would be $(0.01e+f)$ $D = VD^2/15R = 0.1033$ minus an $f_1 = 0.04452$, which results in 0.05878. This value, multiplied by 100 (to convert to percent) and rounded up to the nearest 0.2 of a percent, corresponds to the e value of 6.0 percent. This e value can be found for $R = 1{,}613$ ft at the 50 mph design speed in Table 3-10.

3.3.5 Design Superelevation Tables

Tables 3-8 to 3-12 show minimum values of R for various combinations of superelevation and design speeds for each of five values of maximum superelevation rate (i.e., for a full range of common design conditions). When using one of the tables for a given radius, interpolation is not necessary as the superelevation rate should be determined from a radius equal to, or slightly smaller than, the radius provided in the table. The result is a superelevation rate that is rounded up to the nearest 0.2 of a percent. For example, an 80 km/h [50 mph] curve with a maximum superelevation rate of 8 percent, and a radius of 570 m [1,870 ft], should use the radius of 549 m [1,830 ft] to obtain a superelevation rate of 5.4 percent.

Method 5 was used to distribute e and f for high speeds in calculating the appropriate radius for the range of superelevation rates. A computer program was used to solve Equations 3-9 through 3-22 for the minimum radius using the various combinations of e, f, maximum superelevation, design speed, and running speed (Table 3-6). The minimum radii for each of the five maximum superelevation rates can also be calculated (as shown in Table 3-7) from the simplified curve formula using f values from Figure 3-6.

Table 3-8. Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 4%

Metric									
	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
e (%)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)
NC	163	371	679	951	1310	1740	2170	2640	3250
RC	102	237	441	632	877	1180	1490	1830	2260
2.2	75	187	363	534	749	1020	1290	1590	1980
2.4	51	132	273	435	626	865	1110	1390	1730
2.6	38	99	209	345	508	720	944	1200	1510
2.8	30	79	167	283	422	605	802	1030	1320
3.0	24	64	137	236	356	516	690	893	1150
3.2	20	54	114	199	303	443	597	779	1010
3.4	17	45	96	170	260	382	518	680	879
3.6	14	38	81	144	222	329	448	591	767
3.8	12	31	67	121	187	278	381	505	658
4.0	8	22	47	86	135	203	280	375	492

Note: Use of e_{max} = 4% should be limited to urban conditions.

U.S. Customary										
	V_d = 15 mph	V_d = 20 mph	V_d = 25 mph	V_d = 30 mph	V_d = 35 mph	V_d = 40 mph	V_d = 45 mph	V_d = 50 mph	V_d = 55 mph	V_d = 60 mph
e (%)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)
NC	796	1410	2050	2830	3730	4770	5930	7220	8650	10300
RC	506	902	1340	1880	2490	3220	4040	4940	5950	7080
2.2	399	723	1110	1580	2120	2760	3480	4280	5180	6190
2.4	271	513	838	1270	1760	2340	2980	3690	4500	5410
2.6	201	388	650	1000	1420	1930	2490	3130	3870	4700
2.8	157	308	524	817	1170	1620	2100	2660	3310	4060
3.0	127	251	433	681	982	1370	1800	2290	2860	3530
3.2	105	209	363	576	835	1180	1550	1980	2490	3090
3.4	88	175	307	490	714	1010	1340	1720	2170	2700
3.6	73	147	259	416	610	865	1150	1480	1880	2350
3.8	61	122	215	348	512	730	970	1260	1600	2010
4.0	42	86	154	250	371	533	711	926	1190	1500

Note: Use of e_{max} = 4% should be limited to urban conditions.

Table 3-9. Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 6%

Metric												
	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
e (%)	*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	194	421	738	1050	1440	1910	2360	2880	3510	4060	4770	5240
RC	138	299	525	750	1030	1380	1710	2090	2560	2970	3510	3880
2.2	122	265	465	668	919	1230	1530	1880	2300	2670	3160	3500
2.4	109	236	415	599	825	1110	1380	1700	2080	2420	2870	3190
2.6	97	212	372	540	746	1000	1260	1540	1890	2210	2630	2930
2.8	87	190	334	488	676	910	1150	1410	1730	2020	2420	2700
3.0	78	170	300	443	615	831	1050	1290	1590	1870	2240	2510
3.2	70	152	269	402	561	761	959	1190	1470	1730	2080	2330
3.4	61	133	239	364	511	697	882	1100	1360	1600	1940	2180
3.6	51	113	206	329	465	640	813	1020	1260	1490	1810	2050
3.8	42	96	177	294	422	586	749	939	1170	1390	1700	1930
4.0	36	82	155	261	380	535	690	870	1090	1300	1590	1820
4.2	31	72	136	234	343	488	635	806	1010	1220	1500	1720
4.4	27	63	121	210	311	446	584	746	938	1140	1410	1630
4.6	24	56	108	190	283	408	538	692	873	1070	1330	1540
4.8	21	50	97	172	258	374	496	641	812	997	1260	1470
5.0	19	45	88	156	235	343	457	594	755	933	1190	1400
5.2	17	40	79	142	214	315	421	549	701	871	1120	1330
5.4	15	36	71	128	195	287	386	506	648	810	1060	1260
5.6	13	32	63	115	176	260	351	463	594	747	980	1190
5.8	11	28	56	102	156	232	315	416	537	679	900	1110
6.0	8	21	43	79	123	184	252	336	437	560	756	951

U.S. Customary														
	V_d = 15 mph	V_d = 20 mph	V_d = 25 mph	V_d = 30 mph	V_d = 35 mph	V_d = 40 mph	V_d = 45 mph	V_d = 50 mph	V_d = 55 mph	V_d = 60 mph	V_d = 65 mph	V_d = 70 mph	V_d = 75 mph	V_d = 80 mph
e (%)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)
NC	868	1580	2290	3130	4100	5230	6480	7870	9410	11100	12600	14100	15700	17400
RC	614	1120	1630	2240	2950	3770	4680	5700	6820	8060	9130	10300	11500	12900
2.2	543	991	1450	2000	2630	3370	4190	5100	6110	7230	8200	9240	10400	11600
2.4	482	884	1300	1790	2360	3030	3770	4600	5520	6540	7430	8380	9420	10600
2.6	430	791	1170	1610	2130	2740	3420	4170	5020	5950	6770	7660	8620	9670
2.8	384	709	1050	1460	1930	2490	3110	3800	4580	5440	6200	7030	7930	8910
3.0	341	635	944	1320	1760	2270	2840	3480	4200	4990	5710	6490	7330	8260
3.2	300	566	850	1200	1600	2080	2600	3200	3860	4600	5280	6010	6810	7680
3.4	256	498	761	1080	1460	1900	2390	2940	3560	4250	4890	5580	6340	7180
3.6	209	422	673	972	1320	1740	2190	2710	3290	3940	4540	5210	5930	6720
3.8	176	358	583	864	1190	1590	2010	2490	3040	3650	4230	4860	5560	6320
4.0	151	309	511	766	1070	1440	1840	2300	2810	3390	3950	4550	5220	5950
4.2	131	270	452	684	960	1310	1680	2110	2590	3140	3680	4270	4910	5620
4.4	116	238	402	615	868	1190	1540	1940	2400	2920	3440	4010	4630	5320
4.6	102	212	360	555	788	1090	1410	1780	2210	2710	3220	3770	4380	5040
4.8	91	189	324	502	718	995	1300	1640	2050	2510	3000	3550	4140	4790
5.0	82	169	292	456	654	911	1190	1510	1890	2330	2800	3330	3910	4550
5.2	73	152	264	413	595	833	1090	1390	1750	2160	2610	3120	3690	4320
5.4	65	136	237	373	540	759	995	1280	1610	1990	2420	2910	3460	4090
5.6	58	121	212	335	487	687	903	1160	1470	1830	2230	2700	3230	3840
5.8	51	106	186	296	431	611	806	1040	1320	1650	2020	2460	2970	3560
6.0	39	81	144	231	340	485	643	833	1060	1330	1660	2040	2500	3050

Table 3-10a. Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 8%

Metric												
e (%)	V_d = 20 km/h R (m)	V_d = 30 km/h R (m)	V_d = 40 km/h R (m)	V_d = 50 km/h R (m)	V_d = 60 km/h R (m)	V_d = 70 km/h R (m)	V_d = 80 km/h R (m)	V_d = 90 km/h R (m)	V_d = 100 km/h R (m)	V_d = 110 km/h R (m)	V_d = 120 km/h R (m)	V_d = 130 km/h R (m)
NC	184	443	784	1090	1490	1970	2440	2970	3630	4180	4900	5360
RC	133	322	571	791	1090	1450	1790	2190	2680	3090	3640	4000
2.2	119	288	512	711	976	1300	1620	1980	2420	2790	3290	3620
2.4	107	261	463	644	885	1190	1470	1800	2200	2550	3010	3310
2.6	97	237	421	587	808	1080	1350	1650	2020	2340	2760	3050
2.8	88	216	385	539	742	992	1240	1520	1860	2160	2550	2830
3.0	81	199	354	496	684	916	1150	1410	1730	2000	2370	2630
3.2	74	183	326	458	633	849	1060	1310	1610	1870	2220	2460
3.4	68	169	302	425	588	790	988	1220	1500	1740	2080	2310
3.6	62	156	279	395	548	738	924	1140	1410	1640	1950	2180
3.8	57	144	259	368	512	690	866	1070	1320	1540	1840	2060
4.0	52	134	241	344	479	648	813	1010	1240	1450	1740	1950
4.2	48	124	224	321	449	608	766	948	1180	1380	1650	1850
4.4	43	115	208	301	421	573	722	895	1110	1300	1570	1760
4.6	38	106	192	281	395	540	682	847	1050	1240	1490	1680
4.8	33	96	178	263	371	509	645	803	996	1180	1420	1610
5.0	30	87	163	246	349	480	611	762	947	1120	1360	1540
5.2	27	78	148	229	328	454	579	724	901	1070	1300	1480
5.4	24	71	136	213	307	429	549	689	859	1020	1250	1420
5.6	22	65	125	198	288	405	521	656	819	975	1200	1360
5.8	20	59	115	185	270	382	494	625	781	933	1150	1310
6.0	19	55	106	172	253	360	469	595	746	894	1100	1260
6.2	17	50	98	161	238	340	445	567	713	857	1060	1220
6.4	16	46	91	151	224	322	422	540	681	823	1020	1180
6.6	15	43	85	141	210	304	400	514	651	789	982	1140
6.8	14	40	79	132	198	287	379	489	620	757	948	1100
7.0	13	37	73	123	185	270	358	464	591	724	914	1070
7.2	12	34	68	115	174	254	338	440	561	691	879	1040
7.4	11	31	62	107	162	237	318	415	531	657	842	998
7.6	10	29	57	99	150	221	296	389	499	621	803	962
7.8	9	26	52	90	137	202	273	359	462	579	757	919
8.0	7	20	41	73	113	168	229	304	394	501	667	832

Table 3-10b. Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 8%

U.S. Customary														
	V_d = 15 mph	V_d = 20 mph	V_d = 25 mph	V_d = 30 mph	V_d = 35 mph	V_d = 40 mph	V_d = 45 mph	V_d = 50 mph	V_d = 55 mph	V_d = 60 mph	V_d = 65 mph	V_d = 70 mph	V_d = 75 mph	V_d = 80 mph
e (%)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)
NC	932	1640	2370	3240	4260	5410	6710	8150	9720	11500	12900	14500	16100	17800
RC	676	1190	1720	2370	3120	3970	4930	5990	7150	8440	9510	10700	12000	13300
2.2	605	1070	1550	2130	2800	3570	4440	5400	6450	7620	8600	9660	10800	12000
2.4	546	959	1400	1930	2540	3240	4030	4910	5870	6930	7830	8810	9850	11000
2.6	496	872	1280	1760	2320	2960	3690	4490	5370	6350	7180	8090	9050	10100
2.8	453	796	1170	1610	2130	2720	3390	4130	4950	5850	6630	7470	8370	9340
3.0	415	730	1070	1480	1960	2510	3130	3820	4580	5420	6140	6930	7780	8700
3.2	382	672	985	1370	1820	2330	2900	3550	4250	5040	5720	6460	7260	8130
3.4	352	620	911	1270	1690	2170	2700	3300	3970	4700	5350	6050	6800	7620
3.6	324	572	845	1180	1570	2020	2520	3090	3710	4400	5010	5680	6400	7180
3.8	300	530	784	1100	1470	1890	2360	2890	3480	4140	4710	5350	6030	6780
4.0	277	490	729	1030	1370	1770	2220	2720	3270	3890	4450	5050	5710	6420
4.2	255	453	678	955	1280	1660	2080	2560	3080	3670	4200	4780	5410	6090
4.4	235	418	630	893	1200	1560	1960	2410	2910	3470	3980	4540	5140	5800
4.6	215	384	585	834	1130	1470	1850	2280	2750	3290	3770	4310	4890	5530
4.8	193	349	542	779	1060	1390	1750	2160	2610	3120	3590	4100	4670	5280
5.0	172	314	499	727	991	1310	1650	2040	2470	2960	3410	3910	4460	5050
5.2	154	284	457	676	929	1230	1560	1930	2350	2820	3250	3740	4260	4840
5.4	139	258	420	627	870	1160	1480	1830	2230	2680	3110	3570	4090	4640
5.6	126	236	387	582	813	1090	1390	1740	2120	2550	2970	3420	3920	4460
5.8	115	216	358	542	761	1030	1320	1650	2010	2430	2840	3280	3760	4290
6.0	105	199	332	506	713	965	1250	1560	1920	2320	2710	3150	3620	4140
6.2	97	184	308	472	669	909	1180	1480	1820	2210	2600	3020	3480	3990
6.4	89	170	287	442	628	857	1110	1400	1730	2110	2490	2910	3360	3850
6.6	82	157	267	413	590	808	1050	1330	1650	2010	2380	2790	3240	3720
6.8	76	146	248	386	553	761	990	1260	1560	1910	2280	2690	3120	3600
7.0	70	135	231	360	518	716	933	1190	1480	1820	2180	2580	3010	3480
7.2	64	125	214	336	485	672	878	1120	1400	1720	2070	2470	2900	3370
7.4	59	115	198	312	451	628	822	1060	1320	1630	1970	2350	2780	3250
7.6	54	105	182	287	417	583	765	980	1230	1530	1850	2230	2650	3120
7.8	48	94	164	261	380	533	701	901	1140	1410	1720	2090	2500	2970
8.0	38	76	134	214	314	444	587	758	960	1200	1480	1810	2210	2670

Table 3-11a. Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 10%

Metric												
	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
e (%)	*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	197	454	790	1110	1520	2000	2480	3010	3690	4250	4960	5410
RC	145	333	580	815	1120	1480	1840	2230	2740	3160	3700	4050
2.2	130	300	522	735	1020	1340	1660	2020	2480	2860	3360	3680
2.4	118	272	474	669	920	1220	1520	1840	2260	2620	3070	3370
2.6	108	249	434	612	844	1120	1390	1700	2080	2410	2830	3110
2.8	99	229	399	564	778	1030	1290	1570	1920	2230	2620	2880
3.0	91	211	368	522	720	952	1190	1460	1790	2070	2440	2690
3.2	85	196	342	485	670	887	1110	1360	1670	1940	2280	2520
3.4	79	182	318	453	626	829	1040	1270	1560	1820	2140	2370
3.6	73	170	297	424	586	777	974	1200	1470	1710	2020	2230
3.8	68	159	278	398	551	731	917	1130	1390	1610	1910	2120
4.0	64	149	261	374	519	690	866	1060	1310	1530	1810	2010
4.2	60	140	245	353	490	652	820	1010	1240	1450	1720	1910
4.4	56	132	231	333	464	617	777	953	1180	1380	1640	1820
4.6	53	124	218	315	439	586	738	907	1120	1310	1560	1740
4.8	50	117	206	299	417	557	703	864	1070	1250	1490	1670
5.0	47	111	194	283	396	530	670	824	1020	1200	1430	1600
5.2	44	104	184	269	377	505	640	788	975	1150	1370	1540
5.4	41	98	174	256	359	482	611	754	934	1100	1320	1480
5.6	39	93	164	243	343	461	585	723	896	1060	1270	1420
5.8	36	88	155	232	327	441	561	693	860	1020	1220	1370
6.0	33	82	146	221	312	422	538	666	827	976	1180	1330
6.2	31	77	138	210	298	404	516	640	795	941	1140	1280
6.4	28	72	130	200	285	387	496	616	766	907	1100	1240
6.6	26	67	121	191	273	372	476	593	738	876	1060	1200
6.8	24	62	114	181	261	357	458	571	712	846	1030	1170
7.0	22	58	107	172	249	342	441	551	688	819	993	1130
7.2	21	55	101	164	238	329	425	532	664	792	963	1100
7.4	20	51	95	156	228	315	409	513	642	767	934	1070
7.6	18	48	90	148	218	303	394	496	621	743	907	1040
7.8	17	45	85	141	208	291	380	479	601	721	882	1010
8.0	16	43	80	135	199	279	366	463	582	699	857	981
8.2	15	40	76	128	190	268	353	448	564	679	834	956
8.4	14	38	72	122	182	257	339	432	546	660	812	932
8.6	14	36	68	116	174	246	326	417	528	641	790	910
8.8	13	34	64	110	166	236	313	402	509	621	770	888
9.0	12	32	61	105	158	225	300	386	491	602	751	867
9.2	11	30	57	99	150	215	287	371	472	582	731	847
9.4	11	28	54	94	142	204	274	354	453	560	709	828
9.6	10	26	50	88	133	192	259	337	432	537	685	809
9.8	9	24	46	81	124	179	242	316	407	509	656	786
10.0	7	19	38	68	105	154	210	277	358	454	597	739

Table 3-11b. Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 10%

U.S. Customary														
	V_d = 15 mph	V_d = 20 mph	V_d = 25 mph	V_d = 30 mph	V_d = 35 mph	V_d = 40 mph	V_d = 45 mph	V_d = 50 mph	V_d = 55 mph	V_d = 60 mph	V_d = 65 mph	V_d = 70 mph	V_d = 75 mph	V_d = 80 mph
e (%)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)
NC	947	1680	2420	3320	4350	5520	6830	8280	9890	11700	13100	14700	16300	18000
RC	694	1230	1780	2440	3210	4080	5050	6130	7330	8630	9720	10900	12200	13500
2.2	625	1110	1600	2200	2900	3680	4570	5540	6630	7810	8800	9860	11000	12200
2.4	567	1010	1460	2000	2640	3350	4160	5050	6050	7130	8040	9010	10100	11200
2.6	517	916	1330	1840	2420	3080	3820	4640	5550	6550	7390	8290	9260	10300
2.8	475	841	1230	1690	2230	2840	3520	4280	5130	6050	6840	7680	8580	9550
3.0	438	777	1140	1570	2060	2630	3270	3970	4760	5620	6360	7140	7990	8900
3.2	406	720	1050	1450	1920	2450	3040	3700	4440	5250	5930	6680	7480	8330
3.4	377	670	978	1360	1790	2290	2850	3470	4160	4910	5560	6260	7020	7830
3.6	352	625	913	1270	1680	2150	2670	3250	3900	4620	5230	5900	6620	7390
3.8	329	584	856	1190	1580	2020	2510	3060	3680	4350	4940	5570	6260	6990
4.0	308	547	804	1120	1490	1900	2370	2890	3470	4110	4670	5270	5930	6630
4.2	289	514	756	1060	1400	1800	2240	2740	3290	3900	4430	5010	5630	6300
4.4	271	483	713	994	1330	1700	2120	2590	3120	3700	4210	4760	5370	6010
4.6	255	455	673	940	1260	1610	2020	2460	2970	3520	4010	4540	5120	5740
4.8	240	429	636	890	1190	1530	1920	2340	2830	3360	3830	4340	4900	5490
5.0	226	404	601	844	1130	1460	1830	2240	2700	3200	3660	4150	4690	5270
5.2	213	381	569	802	1080	1390	1740	2130	2580	3060	3500	3980	4500	5060
5.4	200	359	539	762	1030	1330	1660	2040	2460	2930	3360	3820	4320	4860
5.6	188	339	511	724	974	1270	1590	1950	2360	2810	3220	3670	4160	4680
5.8	176	319	484	689	929	1210	1520	1870	2260	2700	3090	3530	4000	4510
6.0	164	299	458	656	886	1160	1460	1790	2170	2590	2980	3400	3860	4360
6.2	152	280	433	624	846	1110	1400	1720	2090	2490	2870	3280	3730	4210
6.4	140	260	409	594	808	1060	1340	1650	2010	2400	2760	3160	3600	4070
6.6	130	242	386	564	772	1020	1290	1590	1930	2310	2670	3060	3480	3940
6.8	120	226	363	536	737	971	1230	1530	1860	2230	2570	2960	3370	3820
7.0	112	212	343	509	704	931	1190	1470	1790	2150	2490	2860	3270	3710
7.2	105	199	324	483	671	892	1140	1410	1730	2070	2410	2770	3170	3600
7.4	98	187	306	460	641	855	1100	1360	1670	2000	2330	2680	3070	3500
7.6	92	176	290	437	612	820	1050	1310	1610	1940	2250	2600	2990	3400
7.8	86	165	274	416	585	786	1010	1260	1550	1870	2180	2530	2900	3310
8.0	81	156	260	396	558	754	968	1220	1500	1810	2120	2450	2820	3220
8.2	76	147	246	377	533	722	930	1170	1440	1750	2050	2380	2750	3140
8.4	72	139	234	359	509	692	893	1130	1390	1690	1990	2320	2670	3060
8.6	68	131	221	341	486	662	856	1080	1340	1630	1930	2250	2600	2980
8.8	64	124	209	324	463	633	820	1040	1290	1570	1870	2190	2540	2910
9.0	60	116	198	307	440	604	784	992	1240	1520	1810	2130	2470	2840
9.2	56	109	186	291	418	574	748	948	1190	1460	1740	2060	2410	2770
9.4	52	102	175	274	395	545	710	903	1130	1390	1670	1990	2340	2710
9.6	48	95	163	256	370	513	671	854	1080	1320	1600	1910	2260	2640
9.8	44	87	150	236	343	477	625	798	1010	1250	1510	1820	2160	2550
10.0	36	72	126	200	292	410	540	694	877	1090	1340	1630	1970	2370

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

Metric												
	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
e (%)	*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
5.2	52	114	201	290	402	535	672	824	1020	1190	1410	1570
5.4	49	108	192	277	384	513	644	790	973	1140	1360	1510
5.6	47	103	183	265	368	492	618	759	936	1100	1310	1460
5.8	45	98	175	254	353	472	594	730	900	1060	1260	1410
6.0	43	94	167	244	339	454	572	703	867	1020	1220	1360
6.2	41	90	159	234	326	436	551	678	837	981	1180	1310
6.4	39	86	153	225	313	420	531	654	808	948	1140	1270
6.6	37	82	146	216	302	405	512	632	781	917	1100	1230
6.8	35	78	140	208	290	391	494	611	755	888	1070	1200
7.0	34	75	134	200	280	377	478	591	731	860	1040	1160
7.2	32	71	128	192	270	364	462	572	708	834	1010	1130
7.4	30	68	122	185	260	352	447	554	686	810	974	1100
7.6	29	65	117	178	251	340	433	537	666	786	947	1070
7.8	27	61	112	172	243	329	420	521	646	764	921	1040
8.0	26	58	107	165	235	319	407	506	628	743	897	1020
8.2	24	55	102	159	227	309	395	491	610	723	874	989
8.4	23	52	97	154	219	299	383	477	593	704	852	965
8.6	22	50	93	148	212	290	372	464	577	686	831	942
8.8	20	47	88	142	205	281	361	451	562	668	811	921
9.0	19	45	85	137	198	273	351	439	547	652	792	900
9.2	18	43	81	132	191	264	341	428	533	636	774	880
9.4	18	41	77	127	185	256	332	416	520	621	756	861
9.6	17	39	74	123	179	249	323	406	507	606	739	843
9.8	16	37	71	118	173	241	314	395	494	592	723	826
10.0	15	36	68	114	167	234	305	385	482	579	708	809
10.2	14	34	65	110	161	226	296	375	471	566	693	793
10.4	14	33	62	105	155	219	288	365	459	553	679	778
10.6	13	31	59	101	150	212	279	355	448	541	665	763
10.8	12	30	57	97	144	204	270	345	436	529	652	749
11.0	12	28	54	93	139	197	261	335	423	516	639	735
11.2	11	27	51	89	133	189	252	324	411	503	626	722
11.4	11	25	49	85	127	182	242	312	397	488	613	709
11.6	10	24	46	80	120	173	232	300	382	472	598	697
11.8	9	22	43	75	113	163	219	285	364	453	579	685
12.0	7	18	36	64	98	143	194	255	328	414	540	665

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

U.S. Customary														
	V_d = 15 mph	V_d = 20 mph	V_d = 25 mph	V_d = 30 mph	V_d = 35 mph	V_d = 40 mph	V_d = 45 mph	V_d = 50 mph	V_d = 55 mph	V_d = 60 mph	V_d = 65 mph	V_d = 70 mph	V_d = 75 mph	V_d = 80 mph
e (%)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)
NC	950	1690	2460	3370	4390	5580	6910	8370	9990	11800	13200	14800	16400	18100
RC	700	1250	1820	2490	3260	4140	5130	6220	7430	8740	9840	11000	12300	13600
2.2	631	1130	1640	2250	2950	3750	4640	5640	6730	7930	8920	9980	11200	12400
2.4	574	1030	1500	2060	2690	3420	4240	5150	6150	7240	8160	9130	10200	11300
2.6	526	936	1370	1890	2470	3140	3900	4730	5660	6670	7510	8420	9380	10500
2.8	484	863	1270	1740	2280	2910	3600	4380	5240	6170	6960	7800	8700	9660
3.0	448	799	1170	1620	2120	2700	3350	4070	4870	5740	6480	7270	8110	9010
3.2	417	743	1090	1510	1970	2520	3130	3800	4550	5370	6060	6800	7600	8440
3.4	389	693	1020	1410	1850	2360	2930	3560	4270	5030	5690	6390	7140	7940
3.6	364	649	953	1320	1730	2220	2750	3350	4020	4740	5360	6020	6740	7500
3.8	341	610	896	1250	1630	2090	2600	3160	3790	4470	5060	5700	6380	7100
4.0	321	574	845	1180	1540	1980	2460	2990	3590	4240	4800	5400	6050	6740
4.2	303	542	798	1110	1460	1870	2330	2840	3400	4020	4560	5130	5750	6420
4.4	286	512	756	1050	1390	1780	2210	2700	3240	3830	4340	4890	5490	6120
4.6	271	485	717	997	1320	1690	2110	2570	3080	3650	4140	4670	5240	5850
4.8	257	460	681	948	1260	1610	2010	2450	2940	3480	3960	4470	5020	5610
5.0	243	437	648	904	1200	1540	1920	2340	2810	3330	3790	4280	4810	5380
5.2	231	415	618	862	1140	1470	1840	2240	2700	3190	3630	4110	4620	5170
5.4	220	395	589	824	1090	1410	1760	2150	2590	3060	3490	3950	4440	4980
5.6	209	377	563	788	1050	1350	1690	2060	2480	2940	3360	3800	4280	4800
5.8	199	359	538	754	1000	1300	1920	1980	2390	2830	3230	3660	4130	4630
6.0	190	343	514	723	960	1250	1560	1910	2300	2730	3110	3530	3990	4470
6.2	181	327	492	694	922	1200	1500	1840	2210	2630	3010	3410	3850	4330
6.4	172	312	471	666	886	1150	1440	1770	2140	2540	2900	3300	3730	4190
6.6	164	298	452	639	852	1110	1390	1710	2060	2450	2810	3190	3610	4060
6.8	156	284	433	615	820	1070	1340	1650	1990	2370	2720	3090	3500	3940
7.0	148	271	415	591	790	1030	1300	1590	1930	2290	2630	3000	3400	3820
7.2	140	258	398	568	762	994	1250	1540	1860	2220	2550	2910	3300	3720
7.4	133	246	382	547	734	960	1210	1490	1810	2150	2470	2820	3200	3610
7.6	125	234	366	527	708	928	1170	1440	1750	2090	2400	2740	3120	3520
7.8	118	222	351	507	684	897	1130	1400	1700	2020	2330	2670	3030	3430
8.0	111	210	336	488	660	868	1100	1360	1650	1970	2270	2600	2950	3340
8.2	105	199	321	470	637	840	1070	1320	1600	1910	2210	2530	2880	3260
8.4	100	190	307	452	615	813	1030	1280	1550	1860	2150	2460	2800	3180
8.6	95	180	294	435	594	787	997	1240	1510	1810	2090	2400	2740	3100
8.8	90	172	281	418	574	762	967	1200	1470	1760	2040	2340	2670	3030
9.0	85	164	270	403	554	738	938	1170	1430	1710	1980	2280	2610	2960
9.2	81	156	259	388	535	715	910	1140	1390	1660	1940	2230	2550	2890
9.4	77	149	248	373	516	693	883	1100	1350	1620	1890	2180	2490	2830
9.6	74	142	238	359	499	671	857	1070	1310	1580	1840	2130	2440	2770
9.8	70	136	228	346	481	650	832	1040	1280	1540	1800	2080	2380	2710
10.0	67	130	219	333	465	629	806	1010	1250	1500	1760	2030	2330	2660
10.2	64	124	210	320	448	608	781	980	1210	1460	1720	1990	2280	2600
10.4	61	118	201	308	432	588	757	951	1180	1430	1680	1940	2240	2550
10.6	58	113	192	296	416	568	732	922	1140	1390	1640	1900	2190	2500
10.8	55	108	184	284	400	548	707	892	1110	1350	1600	1860	2150	2460
11.0	52	102	175	272	384	527	682	862	1070	1310	1560	1820	2110	2410
11.2	49	97	167	259	368	506	656	831	1040	1270	1510	1780	2070	2370
11.4	47	92	158	247	351	485	629	799	995	1220	1470	1730	2020	2320
11.6	44	86	149	233	333	461	600	763	953	1170	1410	1680	1970	2280
11.8	40	80	139	218	312	434	566	722	904	1120	1350	1620	1910	2230
12.0	34	68	119	188	272	381	500	641	807	1000	1220	1480	1790	2130

Under all but extreme weather conditions, vehicles can travel safely at speeds higher than the design speed on horizontal curves with the superelevation rates indicated in the tables. This is due to the development of a radius/superelevation relationship that uses friction factors that are generally considerably less than can be achieved. This is illustrated in Figure 3-5, which compares the friction factors used in design of various types of highway facilities and the maximum side friction factors available on certain wet and dry concrete pavements.

Minimum Radius of Curve for Section with Normal Crown

Very flat horizontal curves need no superelevation. Traffic on the inside lane of a curve has the benefit of some superelevation provided by the normal cross slope. Traffic on the outside lane of a curve has an adverse or negative superelevation due to the normal cross slope, but with flat curves the side friction needed to sustain the lateral acceleration and counteract the negative superelevation is small. However, on successively sharper curves for the same speed, a point is reached where the combination of lateral acceleration and negative superelevation overcomes the allowable side friction, and a positive slope across the entire pavement is desirable to help sustain the lateral acceleration. This condition is the maximum curvature where a crowned pavement cross section is appropriate.

The maximum curvature for normal crowned sections is determined by setting consistently low friction factor values and considering the effect of normal cross slope and both directions of travel. The result is a decreasing degree of curvature for successively higher design speeds.

The term "normal crown" (NC) designates a traveled way cross section used on curves that are so flat that the elimination of adverse cross slope is not needed, and thus the normal cross slope sections can be used. The normal cross slope is generally determined by drainage needs. The term "remove adverse crown" (RC) designates curves where the adverse cross slope should be eliminated by superelevating the entire roadway at the normal cross slope rate.

The usually accepted normal crown rate of cross slope for traveled ways ranges from 1.5 to 2.0 percent. The minimum radius for a normal crown (NC) section for each design speed and maximum superelvation rate is shown in the top row of Tables 3-8 through 3-12. These are curvatures calling for superelevation equal to 1.5 percent—the low range of normal cross slope—and therefore indicate the mathematical limit of a minimally crowned section. Sharper curves should have no adverse cross slopes and be superelevated. For uniformity, these values should be applied to all roadways regardless of the normal cross slope value. The side friction factors developed at these radii because of adverse crown at design speed vary between 0.033 and 0.048. It is evident from their uniform and low value over the range of design speeds and normal cross slopes that these radii are sensible limiting values for normal crown sections.

The 'RC' row in Tables 3-8 through 3-12 presents minimum radii for a computed superelevation rate of 2.0 percent. For curve radii falling between NC and RC, a plane slope across the entire pavement equal to the normal cross slope should typically be used. A transition from the normal crown to a straight-line cross slope will be needed. On a curve sharp enough to need a superelevation rate in excess of 2.0 percent, superelevation should be applied in accordance with Tables 3-8 through 3-12.

3.3.6 Design for Low-Speed Urban Streets

On low-speed urban streets where speed is relatively low and variable, the use of superelevation for horizontal curves can be minimized. Where side friction demand exceeds the assumed available side friction factor for the design speed, superelevation, within the range from the normal cross slope to maximum superelevation, is provided.

Side Friction Factors

Figure 3-6 shows the recommended side friction factors for low-speed streets and highways as a dashed line. These recommended side friction factors provide a reasonable margin of safety at low speeds and lead to somewhat lower superelevation rates as compared to the high-speed friction factors. The side friction factors vary with the design speed from 0.40 at 15 km/h [0.38 at 10 mph] to 0.15 at 70 km/h [45 mph]. A research report (*42*) confirms the appropriateness of these design values.

Superelevation

Although superelevation is beneficial for traffic operations, various factors often combine to make its use impractical in low-speed urban areas. These factors include:

- wide pavement areas;
- the need to meet the grade of adjacent property;
- surface drainage considerations;
- the desire to maintain low-speed operation; and
- frequency of intersecting cross streets, alleys, and driveways.

Therefore, horizontal curves on low-speed urban streets are frequently designed without superelevation, sustaining the lateral force solely with side friction. For traffic traveling along curves to the left, the normal cross slope is an adverse or negative superelevation, but with flat curves the resultant friction needed to sustain the lateral force, even given the negative superelevation, is small.

Where superelevation will be applied to low-speed urban streets, Method 2 is recommended for the design of horizontal curves where, through conditioning, drivers have developed a higher threshold of discomfort. By this method, none of the lateral force is counteracted by superelevation so long as the side friction factor is less than the specified maximum assumed for design for the radius of the curve and the design speed. For sharper curves, f remains at the maximum and e is used in direct proportion to the continued increase in curvature until e reaches e_{max}. The recommended design values for f that are applicable to low-speed streets and highways are shown as a dashed line in Figure 3-6. The radii for the full range of superelevation rates were calculated using Method 2 (i.e., the simplified curve equation) using f values from Figure 3-6 are tabulated in Table 3-13 and graphed in Figure 3-14.

The factors that often make superelevation impractical in low-speed urban areas also make marginal superelevation improvements impractical when reconstructing low-speed urban streets. Therefore, low-speed urban streets may retain their existing cross slope unless the curve has an unacceptable history of curve-related crashes. In such cases, consideration should be given to providing superelevation meeting Table 3-13, and if practical, the superelevation from Tables 3-8 through 3-12.

Table 3-13a. Minimum Radii and Superelevation for Low-Speed Urban Streets

Metric						
	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
e (%)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)
−6.0	11	32	74	151	258	429
−5.0	10	31	70	141	236	386
−4.0	10	30	66	131	218	351
−3.0	10	28	63	123	202	322
−2.8	10	28	62	122	200	316
−2.6	10	28	62	120	197	311
−2.4	10	28	61	119	194	306
−2.2	10	27	61	117	192	301
−2.0	10	27	60	116	189	297
−1.5	9	27	59	113	183	286
0	9	25	55	104	167	257
1.5	9	24	51	96	153	234
2.0	9	24	50	94	149	227
2.2	8	23	50	93	148	224
2.4	8	23	50	92	146	222
2.6	8	23	49	91	145	219
2.8	8	23	49	90	143	217
3.0	8	23	48	89	142	214
3.2	8	23	48	89	140	212
3.4	8	23	48	88	139	210
3.6	8	22	47	87	138	207
3.8	8	22	47	86	136	205
4.0	8	22	47	86	135	203
4.2	8	22	46	85	134	201
4.4	8	22	46	84	132	199
4.6	8	22	46	83	131	197
4.8	8	22	45	83	130	195
5.0	8	21	45	82	129	193
5.2	8	21	45	81	128	191
5.4	8	21	44	81	127	189
5.6	8	21	44	80	125	187
5.8	8	21	44	79	124	185
6.0	8	21	43	79	123	184
6.2	8	21	43	78	122	182
6.4	8	21	43	78	121	180
6.6	8	20	43	77	120	179
6.8	8	20	42	76	119	177
7.0	7	20	42	76	118	175
7.2	7	20	42	75	117	174
7.4	7	20	41	75	116	172
7.6	7	20	41	74	115	171
7.8	7	20	41	73	114	169
8.0	7	20	41	73	113	168
8.2	7	20	40	72	112	166
8.4	7	19	40	72	112	165
8.6	7	19	40	71	111	163
8.8	7	19	40	71	110	162
9.0	7	19	39	70	109	161
9.2	7	19	39	70	108	159
9.4	7	19	39	69	107	158
9.6	7	19	39	69	107	157
9.8	7	19	38	68	106	156
10.0	7	19	38	68	105	154
10.2	7	19	38	67	104	153
10.4	7	18	38	67	103	152
10.6	7	18	37	67	103	151
10.8	7	18	37	66	102	150
11.0	7	18	37	66	101	148
11.2	7	18	37	65	101	147
11.4	7	18	37	65	100	146
11.6	7	18	36	64	99	145
11.8	7	18	36	64	98	144
12.0	7	18	36	64	98	143

Notes:

1. Computed using Superelevation Distribution Method 2.
2. Superelevation may be optional on low-speed urban streets.
3. Negative superelevation values beyond −2.0 percent should be used for unpaved surfaces such as gravel, crushed stone, and earth. However, a normal cross slope of −2.5 percent may be used on paved surfaces in areas with intense rainfall.

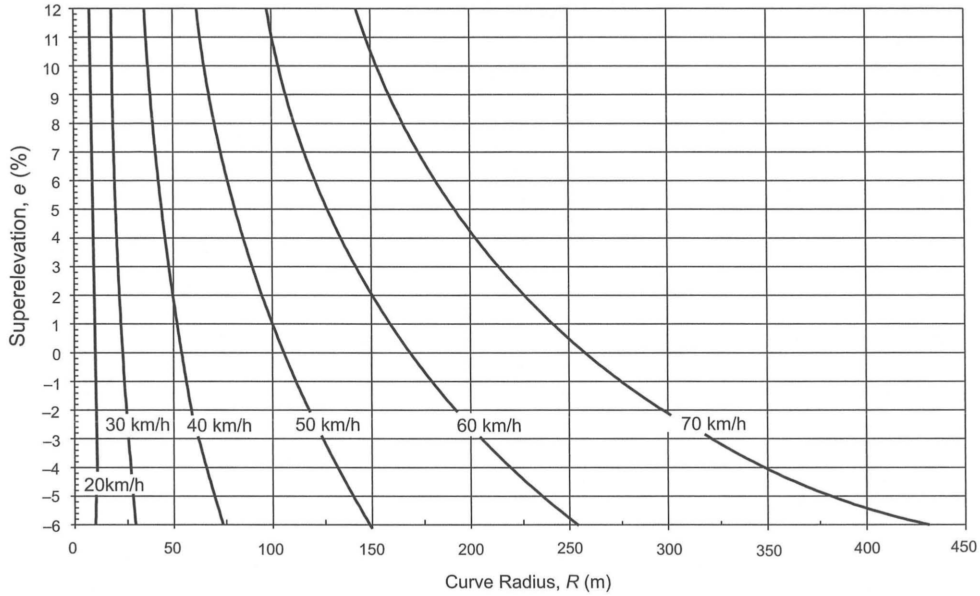
Table 3-13b. Minimum Radii and Superelevation for Low-Speed Urban Streets

U.S. Customary							
	V_d = 15 mph	V_d = 20 mph	V_d = 25 mph	V_d = 30 mph	V_d = 35 mph	V_d = 40 mph	V_d = 45 mph
e (%)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)	*R* (ft)
−6.0	58	127	245	429	681	1067	1500
−5.0	56	121	231	400	628	970	1350
−4.0	54	116	219	375	583	889	1227
−3.0	52	111	208	353	544	821	1125
−2.8	51	110	206	349	537	808	1107
−2.6	51	109	204	345	530	796	1089
−2.4	51	108	202	341	524	784	1071
−2.2	50	108	200	337	517	773	1055
−2.0	50	107	198	333	510	762	1039
−1.5	49	105	194	324	495	736	1000
0	47	99	181	300	454	667	900
1.5	45	94	170	279	419	610	818
2.0	44	92	167	273	408	593	794
2.2	44	91	165	270	404	586	785
2.4	44	91	164	268	400	580	776
2.6	43	90	163	265	396	573	767
2.8	43	89	161	263	393	567	758
3.0	43	89	160	261	389	561	750
3.2	43	88	159	259	385	556	742
3.4	42	88	158	256	382	550	734
3.6	42	87	157	254	378	544	726
3.8	42	87	155	252	375	539	718
4.0	42	86	154	250	371	533	711
4.2	41	85	153	248	368	528	703
4.4	41	85	152	246	365	523	696
4.6	41	84	151	244	361	518	689
4.8	41	84	150	242	358	513	682
5.0	41	83	149	240	355	508	675
5.2	40	83	148	238	352	503	668
5.4	40	82	147	236	349	498	662
5.6	40	82	146	234	346	494	655
5.8	40	81	145	233	343	489	649
6.0	39	81	144	231	340	485	643
6.2	39	80	143	229	337	480	637
6.4	39	80	142	227	335	476	631
6.6	39	79	141	226	332	472	625
6.8	39	79	140	224	329	468	619
7.0	38	78	139	222	327	464	614
7.2	38	78	138	221	324	460	608
7.4	38	78	137	219	322	456	603
7.6	38	77	136	217	319	452	597
7.8	38	77	135	216	317	448	592
8.0	38	76	134	214	314	444	587
8.2	37	76	134	213	312	441	582
8.4	37	75	133	211	309	437	577
8.6	37	75	132	210	307	434	572
8.8	37	74	131	208	305	430	567
9.0	37	74	130	207	302	427	563
9.2	36	74	129	205	300	423	558
9.4	36	73	129	204	298	420	553
9.6	36	73	128	203	296	417	549
9.8	36	72	127	201	294	413	544
10.0	36	72	126	200	292	410	540
10.2	36	72	126	199	290	407	536
10.4	35	71	125	197	288	404	531
10.6	35	71	124	196	286	401	527
10.8	35	71	123	195	284	398	523
11.0	35	70	123	194	282	395	519
11.2	35	70	122	192	280	392	515
11.4	35	69	121	191	278	389	511
11.6	34	69	120	190	276	386	508
11.8	34	69	120	189	274	384	504
12.0	34	68	119	188	272	381	500

Notes:

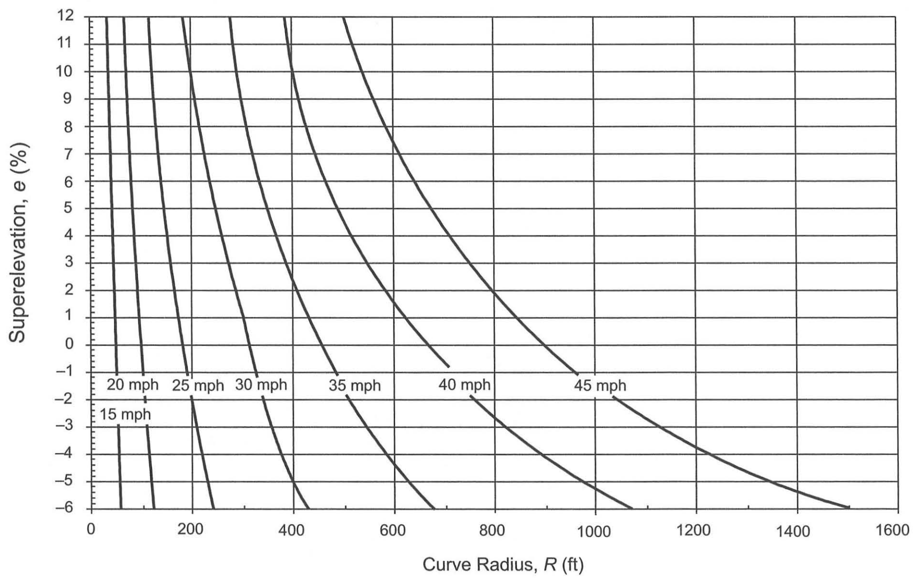
1. Computed using Superelevation Distribution Method 2.
2. Superelevation may be optional on low-speed urban streets.
3. Negative superelevation values beyond −2.0 percent should be used for unpaved surfaces such as gravel, crushed stone, and earth. However, a normal cross slope of −2.5 percent may be used on paved surfaces in areas with intense rainfall.

METRIC



Note: Negative superelevation values beyond –2.0 percent should be used for unpaved surfaces such as gravel, crushed stone, and earth. However, areas with intense rainfall may use normal cross slopes of –2.5 percent on paved surfaces.

U.S. CUSTOMARY



Note: Negative superelevation values beyond –2.0 percent should be used for unpaved surfaces such as gravel, crushed stone, and earth. However, areas with intense rainfall may use normal cross slopes of –2.5 percent on paved surfaces.

Figure 3-14. Superelevation, Radius, and Design Speed for Low-Speed Urban Street Design

Sharpest Curve without Superelevation Minimum Radius for Section with Normal Crown

The –2.0 percent row in Table 3-13 provides the minimum curve radii for which a normal crown of 2.0 percent should be retained. Likewise, the –1.5 percent row provides the minimum curve radii for which a normal crown of 1.5 percent should be retained. Sharper curves should have no adverse cross slope and should be superelevated in accordance with Table 3-13.

3.3.7 Turning Roadways

Turning roadways include interchange ramps and intersection curves for right-turning vehicles. Loop or diamond configurations for turning roadways are commonly used at interchanges and consist of combinations of tangents and curves. At intersections, turning roadways have a diamond configuration and consist of curves (often compound curves).

The minimum radii used for design should preferably be measured from the inner edge of the traveled way rather than the middle of the vehicle path or the centerline of the traveled way. The radius and corresponding superelevation rate for turning roadways is determined on the basis of the design speed and the values in Tables 3-8 through 3-12. These tables use the Method 5 superelevation distribution method discussed previously and provide additional superelevation for turning roadways with radii greater than the minimum radius for the design speed and selected maximum superelevation rate.

In selecting a minimum radius, it is recognized that sharper curves, having shorter lengths, provide less opportunity for developing a large rate of superelevation. This condition applies particularly to intersections where the turning roadway is often close to the intersection proper, where much of its area is adjacent to the through traveled way, and where the complete turn is made through a total angle of about 90 degrees.

Turning roadway design does not apply to the design for turns at intersections without separate turning roadways. Refer to Chapter 9 for the design of intersections, including the use of compound curves to accommodate the inside edge of the design vehicle's swept path.

Design Speed

As further discussed in Chapter 9, vehicles turning at intersections designed for minimum-radius turns have to operate at low speed, perhaps less than 15 km/h [10 mph]. While it is desirable and often practical to design for turning vehicles operating at higher speeds, it is often appropriate for safety and economy to use lower turning speeds at most intersections. The speeds for which these intersection curves should be designed depend on vehicle speeds on the approach highways, the type of intersection, and the volumes of through and turning traffic. Generally, a desirable turning speed for design is the average running speed of traffic on the highway approaching the turn. Designs at such speeds offer little hindrance to smooth flow of traffic and may be justified for some interchange ramps or, at intersections, for certain movements that involve little or no conflict with pedestrians or other vehicular traffic.

Maximum Superelevation for Turning Roadways

Turning roadways include interchange ramps and intersection curves for right-turning vehicles. As much superelevation as practical, up to a maximum value, should be developed on ramps to counter skidding and overturning.

At the terminal of the turning roadway where all traffic comes to a stop, as at stop signs, a lesser amount of superelevation is usually appropriate. Also where a significant number of large trucks will be using right-turning roadways at intersections, flatter curves that need less superelevation should be provided because large trucks may have trouble negotiating intersection curves with superelevation. This is particularly true where trucks cross over from a roadway or ramp sloping in one direction to one sloping the other way. Superelevation for curves on turning roadways at intersections is further discussed under that heading in Section 9.6.6.

Use of Compound Curves

When the design speed of the turning roadway is 70 km/h [45 mph] or less, compound curvature can be used to form the entire alignment of the turning roadway. When the design speed exceeds 70 km/h [45 mph], the exclusive use of compound curves is often impractical, as it tends to need a large amount of right-of-way. Thus, high-speed turning roadways follow the interchange ramp design guidelines in Section 10.9.6 and include a mix of tangents and curves. By this approach, the design can be more sensitive to right-of-way impacts as well as to driver comfort and safety.

An important consideration is to avoid compound curve designs that mislead the motorist's expectation of how sharp the curve radius is. For compound curves on turning roadways, it is preferable that the ratio of the flatter radius to the sharper radius not exceed 2:1. This ratio results in a reduction of approximately 10 km/h [6 mph] in average running speeds for the two curves.

Curves that are compounded should not be too short or their effect in enabling a change in speed from the tangent or flat curve to the sharp curve is lost. In a series of curves of decreasing radii, each curve should be long enough to enable the driver to decelerate at a reasonable rate. At intersections, a maximum deceleration rate of 5 km/h/s [3 mph/s] may be used (although 3 km/h/s [2 mph/s] is desirable). The desirable rate represents very light braking, because deceleration in gear alone generally results in overall rates between 1.5 and 2.5 km/h/s [1 and 1.5 mph/s]. Minimum compound curve lengths based on these criteria are presented in Table 3-14.

The compound curve lengths in Table 3-14 are developed on the premise that travel is in the direction of sharper curvature. For the acceleration condition, the 2:1 ratio is not as critical and may be exceeded.

Table 3-14. Lengths of Circular Arcs for Different Compound Curve Radii

Metric			U.S. Customary		
	Minimum Length of Circular Arc (m)			Minimum Length of Circular Arc (ft)	
Radius (m)	Acceptable	Desirable	Radius (ft)	Acceptable	Desirable
30	12	20	100	40	60
50	15	20	150	50	70
60	20	30	200	60	90
75	25	35	250	80	120
100	30	45	300	100	140
125	35	55	400	120	180
150 or more	45	60	500 or more	140	200

3.3.8 Transition Design Controls

General Considerations

The design of transition sections includes consideration of transitions in the roadway cross slope and possible transition curves incorporated in the horizontal alignment. The former consideration is referred to as superelevation transition and the latter is referred to as alignment transition. Where both transition components are used, they occur together over a common section of roadway at the beginning and end of the main line circular curves.

The superelevation transition section consists of the superelevation runoff and tangent runout sections. The superelevation runoff section consists of the length of roadway needed to accomplish a change in outside-lane cross slope from zero (flat) to full superelevation, or vice versa. The tangent runout section consists of the length of roadway needed to accomplish a change in outside-lane cross slope from the normal cross slope rate to zero (flat), or vice versa. To limit lateral acceleration, the pavement rotation in the superelevation transition section should be achieved over a length that is sufficient to make such rotation imperceptible to drivers. To be pleasing in appearance, the pavement edges should not appear distorted to the driver.

In the alignment transition section, a spiral or compound transition curve may be used to introduce the main circular curve in a natural manner (i.e., one that is consistent with the driver's steered path). Such transition curvature consists of one or more curves aligned and located to provide a gradual change in alignment radius. As a result, an alignment transition gently introduces the lateral acceleration associated with the curve. While such a gradual change in path and lateral acceleration is appealing, there is no definitive evidence that transition curves are essential to the safe operation of the roadway and, as a result, they are not used by many agencies.

When a transition curve is not used, the roadway tangent directly adjoins the main circular curve. This type of transition design is referred to as the "tangent-to-curve" transition.

Some agencies employ spiral curves and use their length to make the appropriate superelevation transition. A spiral curve approximates the natural turning path of a vehicle. One agency believes that the length of spiral should be based on a 4-s minimum maneuver time at the design speed of the highway. Other agencies do not employ spiral curves but empirically designate proportional lengths of tangent and circular curve for the same purpose. In either case, as far as can be determined, the length of roadway to achieve the superelevation runoff should be the same.

Review of current design practice indicates that the length of a superelevation runoff section is largely governed by its appearance. Spiral transition curve lengths determined by other factors are often shorter than those determined for general appearance. Therefore, theoretically derived spiral lengths are replaced with longer empirically derived runoff lengths. A number of agencies have established one or more control runoff lengths within a range of 30 to 200 m [100 to 650 ft], but there is no universally accepted empirical basis for determining runoff length, considering all likely traveled way widths. In one widely used empirical expression, the runoff length is determined as a function of the slope of the outside edge of the traveled way relative to the centerline profile.

Tangent-to-Curve Transition

Minimum length of superelevation runoff—For appearance and comfort, the length of superelevation runoff should be based on a maximum acceptable difference between the longitudinal grades of the axis of rotation and the edge of pavement. The axis of rotation is generally represented by the alignment centerline for undivided roadways; however, other pavement reference lines can be used. These lines and the rationale for their use is discussed later in a subsection titled, "Methods of Attaining Superelevation."

Current practice is to limit the grade difference, referred to as the relative gradient, to a maximum value of 0.50 percent or a longitudinal slope of 1:200 at 80 km/h [50 mph]. In one source (*63*), this same 1:200 slope is used for a design speed of 80 km/h [50 mph] and higher. Where design speeds are less than 80 km/h [50 mph], greater relative slopes are used. To reflect the importance of the higher design speed and to harmonize with the flatter curving elements, both horizontal and vertical, it appears logical to extrapolate the relative slopes for the higher design speeds.

The maximum relative gradient is varied with design speed to provide longer runoff lengths at higher speeds and shorter lengths at lower speeds. Experience indicates that relative gradients of 0.80 and 0.35 percent [0.78 and 0.35 percent] provide acceptable runoff lengths for design speeds of 20 and 130 km/h [15 and 80 mph], respectively.

Interpolation between these values provides the maximum relative gradients shown in Table 3-15. The maximum relative gradient between profiles of the edges of two-lane traveled ways should be double those given in the table. Runoff lengths determined on this basis are directly proportional to the total superelevation, which is the product of the lane width and superelevation rate.

Previous editions of this policy have suggested that runoff lengths should be at least equal to the distance traveled in 2.0 s at the design speed. This criterion tended to determine the runoff lengths of curves with small superelevation rates, high speed, or both. Experience with the 2.0-s criterion indicates that the improvement in appearance is outweighed by a tendency to aggravate problems associated with pavement drainage in the transition section. In fact, it is noted that some agencies do not use this control. From this evidence, it is concluded that a comfortable and aesthetically pleasing runoff design can be attained through the exclusive use of the maximum relative gradient criterion.

Table 3-15. Maximum Relative Gradients

Metric			U.S. Customary		
Design Speed (km/h)	Maximum Relative Gradient (%)	Equivalent Maximum Relative Slope	Design Speed (mph)	Maximum Relative Gradient (%)	Equivalent Maximum Relative Slope
20	0.80	1:125	15	0.78	1:128
30	0.75	1:133	20	0.74	1:135
40	0.70	1:143	25	0.70	1:143
50	0.65	1:154	30	0.66	1:152
60	0.60	1:167	35	0.62	1:161
70	0.55	1:182	40	0.58	1:172
80	0.50	1:200	45	0.54	1:185
90	0.47	1:213	50	0.50	1:200
100	0.44	1:227	55	0.47	1:213
110	0.41	1:244	60	0.45	1:222
120	0.38	1:263	65	0.43	1:233
130	0.35	1:286	70	0.40	1:250
			75	0.38	1:263
			80	0.35	1:286

On the basis of the preceding discussion, the minimum length of runoff should be determined as:

Metric

$$L_r = \frac{(wn_1)\,e_d}{\Delta}(b_w) \tag{3-23}$$

where:

L_r = minimum length of superelevation runoff, m

w = width of one traffic lane, m (typically 3.6 m)

n_1 = number of lanes rotated

e_d = design superelevation rate, percent

b_w = adjustment factor for number of lanes rotated

Δ = maximum relative gradient, percent

U.S. Customary

$$L_r = \frac{(wn_1)\,e_d}{\Delta}(b_w) \tag{3-23}$$

where:

L_r = minimum length of superelevation runoff, ft

w = width of one traffic lane, ft (typically 12 ft)

n_1 = number of lanes rotated

e_d = design superelevation rate, percent

b_w = adjustment factor for number of lanes rotated

Δ = maximum relative gradient, percent

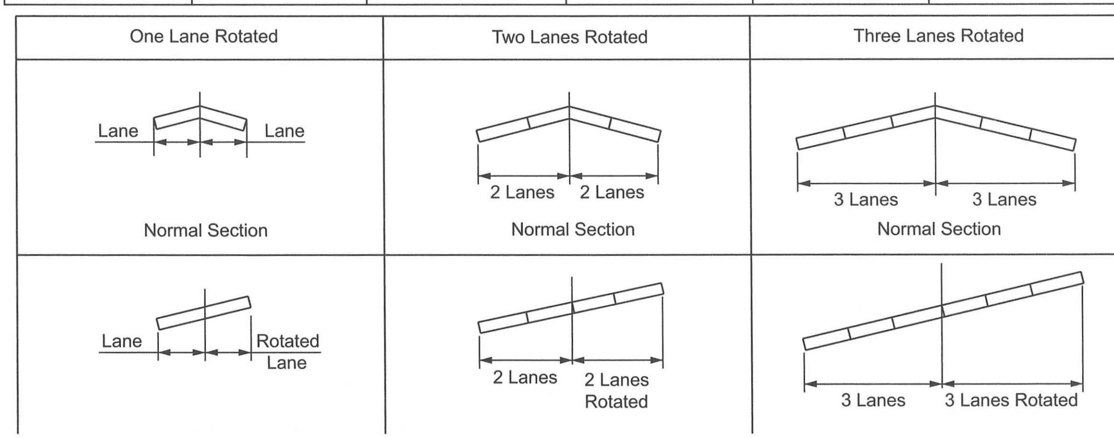
Equation 3-23 can be used directly for undivided streets or highways where the cross section is rotated about the highway centerline and n_1 is equal to one-half the number of lanes in the cross section. More generally, Equation 3-23 can be used for rotation about any pavement reference line provided that the rotated width (wn_1) has a common superelevation rate and is rotated as a plane.

A strict application of the maximum relative gradient criterion provides runoff lengths for four-lane undivided roadways that are double those for two-lane roadways; those for six-lane undivided roadways would be tripled. While lengths of this order may be considered desirable, it is often not practical to provide such lengths in design. On a purely empirical basis, it is recommended that minimum superelevation runoff lengths be adjusted downward to avoid excessive lengths for multilane roadways. The recommended adjustment factors are presented in Table 3-16.

The adjustment factors listed in Table 3-16 are directly applicable to undivided streets and highways. Development of runoff for divided highways is discussed in more detail later in the subsection titled, "Axis of Rotation with a Median." The topic of runoff superelevation for turning roadway designs at intersections and through interchanges is discussed in Chapters 9 and 10, respectively.

Table 3-16. Adjustment Factor for Number of Lanes Rotated

Metric			U.S. Customary		
Number of Lanes Rotated, n_1	Adjustment Factor,* b_w	Length Increase Relative to One-Lane Rotated, $(= n_1 b_w)$	Number of Lanes Rotated, n_1	Adjustment Factor,* b_w	Length Increase Relative to One-Lane Rotated, $(= n_1 b_w)$
1	1.00	1.0	1	1.00	1.0
1.5	0.83	1.25	1.5	0.83	1.25
2	0.75	1.5	2	0.75	1.5
2.5	0.70	1.75	2.5	0.70	1.75
3	0.67	2.0	3	0.67	2.0
3.5	0.64	2.25	3.5	0.64	2.25



* $b_w = [1 + 0.5 (n_1 - 1)]/n_1$

Typical minimum superelevation runoff lengths are presented in Table 3-17. The lengths shown represent cases where one or two lanes are rotated about a pavement edge. The former case is found on two-lane roadways where the pavement is rotated about the centerline or on one-lane interchange ramps where the pavement rotation is about an edge line. The latter case is found on multilane undivided roadways where each direction is separately rotated about an edge line.

Elimination of the 2.0-s travel-time criterion previously discussed results in shorter runoff lengths for smaller superelevation rates and higher speeds. However, even the shortest runoff lengths (corresponding to a superelevation rate of 2.0 percent) correspond to travel times of 0.6 s, which is sufficient to provide a smooth edge-of-pavement profile.

For high-type alignments, superelevation runoff lengths longer than those shown in Table 3-17 may be desirable. In this case, drainage needs or the desire for smoothness in the traveled-way-edge profiles may call for a small increase in runoff length.

The superelevation runoff lengths given in Table 3-17 are based on 3.6-m [12-ft] lanes. For other lane widths, the appropriate runoff length should vary in proportion to the ratio of the actual lane width to 3.6 m [12 ft]. Shorter lengths could be applied for designs with 3.0- and 3.3-m [10- and 11-ft] lanes, but considerations of consistency and practicality suggest that the runoff lengths for 3.6-m [12-ft] lanes should be used in all cases.

Table 3-17a. Superelevation Runoff L_r (m) for Horizontal Curves

Metric																								
	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	
	Number of Lanes Rotated. Note that 1 lane rotated is typical for a 2-lane highway, 2 lanes rotated is typical for a 4-lane highway, etc. (See Table 3-16.)																							
	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
e (%)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)
1.5	7	10	7	11	8	12	8	13	9	14	10	15	11	16	12	17	12	18	13	20	14	21	15	23
2.0	9	14	10	14	10	15	11	17	12	18	13	20	14	22	15	23	16	25	18	26	19	28	21	31
2.2	10	15	11	16	11	17	12	18	13	20	14	22	16	24	17	25	18	27	19	29	21	31	23	34
2.4	11	16	12	17	12	19	13	20	14	22	16	24	17	26	18	28	20	29	21	32	23	34	25	37
2.6	12	18	12	19	13	20	14	22	16	23	17	26	19	28	20	30	21	32	23	34	25	37	27	40
2.8	13	19	13	20	14	22	16	23	17	25	18	27	20	30	21	32	23	34	25	37	27	40	29	43
3.0	14	20	14	22	15	23	17	25	18	27	20	29	22	32	23	34	25	37	26	40	28	43	31	46
3.2	14	22	15	23	16	25	18	27	19	29	21	31	23	35	25	37	26	39	28	42	30	45	33	49
3.4	15	23	16	24	17	26	19	28	20	31	22	33	24	37	26	39	28	42	30	45	32	48	35	52
3.6	16	24	17	26	19	28	20	30	22	32	24	35	26	39	28	41	29	44	32	47	34	51	37	56
3.8	17	26	18	27	20	29	21	32	23	34	25	37	27	41	29	44	31	47	33	50	36	54	39	59
4.0	18	27	19	29	21	31	22	33	24	36	26	39	29	43	31	46	33	49	35	53	38	57	41	62
4.2	19	28	20	30	22	32	23	35	25	38	27	41	30	45	32	48	34	52	37	55	40	60	43	65
4.4	20	30	21	32	23	34	24	37	26	40	29	43	32	48	34	51	36	54	39	58	42	63	45	68
4.6	21	31	22	33	24	35	25	38	28	41	30	45	33	50	35	53	38	56	40	61	44	65	47	71
4.8	22	32	23	35	25	37	27	40	29	43	31	47	35	52	37	55	39	59	42	63	45	68	49	74
5.0	23	34	24	36	26	39	28	42	30	45	33	49	36	54	38	57	41	61	44	66	47	71	51	77
5.2	23	35	25	37	27	40	29	43	31	47	34	51	37	56	40	60	43	64	46	68	49	74	53	80
5.4	24	36	26	39	28	42	30	45	32	49	35	53	39	58	41	62	44	66	47	71	51	77	56	83
5.6	25	38	27	40	29	43	31	47	34	50	37	55	40	60	43	64	46	69	49	74	53	80	58	86
5.8	26	39	28	42	30	45	32	48	35	52	38	57	42	63	44	67	47	71	51	76	55	82	60	89
6.0	27	41	29	43	31	46	33	50	36	54	39	59	43	65	46	69	49	74	53	79	57	85	62	93
6.2	28	42	30	45	32	48	34	52	37	56	41	61	45	67	47	71	51	76	54	82	59	88	64	96
6.4	29	43	31	46	33	49	35	53	38	58	42	63	46	69	49	74	52	79	56	84	61	91	66	99
6.6	30	45	32	48	34	51	37	55	40	59	43	65	48	71	51	76	54	81	58	87	63	94	68	102
6.8	31	46	33	49	35	52	38	56	41	61	45	67	49	73	52	78	56	83	60	90	64	97	70	105
7.0	31	47	34	50	36	54	39	58	42	63	46	69	50	76	54	80	57	86	61	92	66	99	72	108
7.2	32	49	35	52	37	56	40	60	43	65	47	71	52	78	55	83	59	88	63	95	68	102	74	111
7.4	33	50	36	53	38	57	41	61	44	67	48	73	53	80	57	85	61	91	65	97	70	105	76	114
7.6	34	51	36	55	39	59	42	63	46	68	50	75	55	82	58	87	62	93	67	100	72	108	78	117
7.8	35	53	37	56	40	60	43	65	47	70	51	77	56	84	60	90	64	96	68	103	74	111	80	120
8.0	36	54	38	58	41	62	44	66	48	72	52	79	58	86	61	92	65	98	70	105	76	114	82	123
8.2	37	55	39	59	42	63	45	68	49	74	54	81	59	89	63	94	67	101	72	108	78	117	84	127
8.4	38	57	40	60	43	65	47	70	50	76	55	82	60	91	64	97	69	103	74	111	80	119	86	130
8.6	39	58	41	62	44	66	48	71	52	77	56	84	62	93	66	99	70	106	76	113	81	122	88	133
8.8	40	59	42	63	45	68	49	73	53	79	58	86	63	95	67	101	72	108	77	116	83	125	91	136
9.0	40	61	43	65	46	69	50	75	54	81	59	88	65	97	69	103	74	110	79	119	85	128	93	139
9.2	41	62	44	66	47	71	51	76	55	83	60	90	66	99	70	106	75	113	81	121	87	131	95	142
9.4	42	63	45	68	48	73	52	78	56	85	62	92	68	102	72	108	77	115	83	124	89	134	97	145
9.6	43	65	46	69	49	74	53	80	58	86	63	94	69	104	74	110	79	118	84	126	91	136	99	148
9.8	44	66	47	71	50	76	54	81	59	88	64	96	71	106	75	113	80	120	86	129	93	139	101	151
10.0	45	68	48	72	51	77	55	83	60	90	65	98	72	108	77	115	82	123	88	132	95	142	103	154
10.2	46	69	49	73	52	79	56	85	61	92	67	100	73	110	78	117	83	125	90	134	97	145	105	157
10.4	47	70	50	75	53	80	58	86	62	94	68	102	75	112	80	119	85	128	91	137	99	148	107	160
10.6	48	72	51	76	55	82	59	88	64	95	69	104	76	114	81	122	87	130	93	140	100	151	109	164
10.8	49	73	52	78	56	83	60	90	65	97	71	106	78	117	83	124	88	133	95	142	102	153	111	167
11.0	50	74	53	79	57	85	61	91	66	99	72	108	79	119	84	126	90	135	97	145	104	156	113	170
11.2	50	76	54	81	58	86	62	93	67	101	73	110	81	121	86	129	92	137	98	148	106	159	115	173
11.4	51	77	55	82	59	88	63	95	68	103	75	112	82	123	87	131	93	140	100	150	108	162	117	176
11.6	52	78	56	84	60	89	64	96	70	104	76	114	84	125	89	133	95	142	102	153	110	165	119	179
11.8	53	80	57	85	61	91	65	98	71	106	77	116	85	127	90	136	97	145	104	155	112	168	121	182
12.0	54	81	58	86	62	93	66	100	72	108	79	118	86	130	92	138	98	147	105	158	114	171	123	185

Table 3-17b. Superelevation Runoff L_r (ft) for Horizontal Curves

U.S. Customary																												
	V_d = 15 mph		V_d = 20 mph		V_d = 25 mph		V_d = 30 mph		V_d = 35 mph		V_d = 40 mph		V_d = 45 mph		V_d = 50 mph		V_d = 55 mph		V_d = 60 mph		V_d = 65 mph		V_d = 70 mph		V_d = 75 mph		V_d = 80 mph	
	Number of Lanes Rotated. Note that 1 lane rotated is typical for a 2-lane highway, 2 lanes rotated is typical for a 4-lane highway, etc. (See Table 3-16.)																											
	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
e (%)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)	L_r (ft)
1.5	23	35	24	37	26	39	27	41	29	44	31	47	33	50	36	54	38	58	40	60	42	63	45	68	47	71	52	77
2.0	31	46	32	49	34	51	36	55	39	58	41	62	44	67	48	72	51	77	53	80	56	84	60	90	63	95	69	103
2.2	34	51	36	54	38	57	40	60	43	64	46	68	49	73	53	79	56	84	59	88	61	92	66	99	69	104	75	113
2.4	37	55	39	58	41	62	44	65	46	70	50	74	53	80	58	86	61	92	64	96	67	100	72	108	76	114	82	123
2.6	40	60	42	63	45	67	47	71	50	75	54	81	58	87	62	94	66	100	69	104	73	109	78	117	82	123	89	134
2.8	43	65	45	68	48	72	51	76	54	81	58	87	62	93	67	101	71	107	75	112	78	117	84	126	88	133	96	144
3.0	46	69	49	73	51	77	55	82	58	87	62	93	67	100	72	108	77	115	80	120	84	126	90	135	95	142	103	154
3.2	49	74	52	78	55	82	58	87	62	93	66	99	71	107	77	115	82	123	85	128	89	134	96	144	101	152	110	165
3.4	52	78	55	83	58	87	62	93	66	99	70	106	76	113	82	122	87	130	91	136	95	142	102	153	107	161	117	175
3.6	55	83	58	88	62	93	65	98	70	105	74	112	80	120	86	130	92	138	96	144	100	151	108	162	114	171	123	185
3.8	58	88	62	92	65	98	69	104	74	110	79	118	84	127	91	137	97	146	101	152	106	159	114	171	120	180	130	195
4.0	62	92	65	97	69	103	73	109	77	116	83	124	89	133	96	144	102	153	107	160	112	167	120	180	126	189	137	206
4.2	65	97	68	102	72	108	76	115	81	122	87	130	93	140	101	151	107	161	112	168	117	176	126	189	133	199	144	216
4.4	68	102	71	107	75	113	80	120	85	128	91	137	98	147	106	158	112	169	117	176	123	184	132	198	139	208	151	226
4.6	71	106	75	112	79	118	84	125	89	134	95	143	102	153	110	166	117	176	123	184	128	193	138	207	145	218	158	237
4.8	74	111	78	117	82	123	87	131	93	139	99	149	107	160	115	173	123	184	128	192	134	201	144	216	152	227	165	247
5.0	77	115	81	122	86	129	91	136	97	145	103	155	111	167	120	180	128	191	133	200	140	209	150	225	158	237	171	257
5.2	80	120	84	126	89	134	95	142	101	151	108	161	116	173	125	187	133	199	139	208	145	218	156	234	164	246	178	267
5.4	83	125	88	131	93	139	98	147	105	157	112	168	120	180	130	194	138	207	144	216	151	226	162	243	171	256	185	278
5.6	86	129	91	136	96	144	102	153	108	163	116	174	124	187	134	202	143	214	149	224	156	234	168	252	177	265	192	288
5.8	89	134	94	141	99	149	105	158	112	168	120	180	129	193	139	209	148	222	155	232	162	243	174	261	183	275	199	298
6.0	92	138	97	146	103	154	109	164	116	174	124	186	133	200	144	216	153	230	160	240	167	251	180	270	189	284	206	309
6.2	95	143	101	151	106	159	113	169	120	180	128	192	138	207	149	223	158	237	165	248	173	260	186	279	196	294	213	319
6.4	98	148	104	156	110	165	116	175	124	186	132	199	142	213	154	230	163	245	171	256	179	268	192	288	202	303	219	329
6.6	102	152	107	161	113	170	120	180	128	192	137	205	147	220	158	238	169	253	176	264	184	276	198	297	208	313	226	339
6.8	105	157	110	165	117	175	124	185	132	197	141	211	151	227	163	245	174	260	181	272	190	285	204	306	215	322	233	350
7.0	108	162	114	170	120	180	127	191	135	203	145	217	156	233	168	252	179	268	187	280	195	293	210	315	221	332	240	360
7.2	111	166	117	175	123	185	131	196	139	209	149	223	160	240	173	259	184	276	192	288	201	301	216	324	227	341	247	370
7.4	114	171	120	180	127	190	135	202	143	215	153	230	164	247	178	266	189	283	197	296	207	310	222	333	234	351	254	381
7.6	117	175	123	185	130	195	138	207	147	221	157	236	169	253	182	274	194	291	203	304	212	318	228	342	240	360	261	391
7.8	120	180	126	190	134	201	142	213	151	226	161	242	173	260	187	281	199	299	208	312	218	327	234	351	246	369	267	401
8.0	123	185	130	195	137	206	145	218	155	232	166	248	178	267	192	288	204	306	213	320	223	335	240	360	253	379	274	411
8.2	126	189	133	199	141	211	149	224	159	238	170	254	182	273	197	295	209	314	219	328	229	343	246	369	259	388	281	422
8.4	129	194	136	204	144	216	153	229	163	244	174	261	187	280	202	302	214	322	224	336	234	352	252	378	265	398	288	432
8.6	132	198	139	209	147	221	156	235	166	250	178	267	191	287	206	310	220	329	229	344	240	360	258	387	272	407	295	442
8.8	135	203	143	214	151	226	160	240	170	255	182	273	196	293	211	317	225	337	235	352	246	368	264	396	278	417	302	453
9.0	138	208	146	219	154	231	164	245	174	261	186	279	200	300	216	324	230	345	240	360	251	377	270	405	284	426	309	463
9.2	142	212	149	224	158	237	167	251	178	267	190	286	204	307	221	331	235	352	245	368	257	385	276	414	291	436	315	473
9.4	145	217	152	229	161	242	171	256	182	273	194	292	209	313	226	338	240	360	251	376	262	393	282	423	297	445	322	483
9.6	148	222	156	234	165	247	175	262	186	279	199	298	213	320	230	346	245	368	256	384	268	402	288	432	303	455	329	494
9.8	151	226	159	238	168	252	178	267	190	285	203	304	218	327	235	353	250	375	261	392	273	410	294	441	309	464	336	504
10.0	154	231	162	243	171	257	182	273	194	290	207	310	222	333	240	360	255	383	267	400	279	419	300	450	316	474	343	514
10.2	157	235	165	248	175	262	185	278	197	296	211	317	227	340	245	367	260	391	272	408	285	427	306	459	322	483	350	525
10.4	160	240	169	253	178	267	189	284	201	302	215	323	231	347	250	374	266	398	277	416	290	435	312	468	328	493	357	535
10.6	163	245	172	258	182	273	193	289	205	308	219	329	236	353	254	382	271	406	283	424	296	444	318	477	335	502	363	545
10.8	166	249	175	263	185	278	196	295	209	314	223	335	240	360	259	389	276	414	288	432	301	452	324	486	341	512	370	555
11.0	169	254	178	268	189	283	200	300	213	319	228	341	244	367	264	396	281	421	293	440	307	460	330	495	347	521	377	566
11.2	172	258	182	272	192	288	204	305	217	325	232	348	249	373	269	403	286	429	299	448	313	469	336	504	354	531	384	576
11.4	175	263	185	277	195	293	207	311	221	331	236	354	253	380	274	410	291	437	304	456	318	477	342	513	360	540	391	586
11.6	178	268	188	282	199	298	211	316	225	337	240	360	258	387	278	418	296	444	309	464	324	486	348	522	366	549	398	597
11.8	182	272	191	287	202	303	215	322	228	343	244	366	262	393	283	425	301	452	315	472	329	494	354	531	373	559	405	607
12.0	185	277	195	292	206	309	218	327	232	348	248	372	267	400	288	432	306	460	320	480	335	502	360	540	379	568	411	617

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:

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

(3-24)

The tangent runout lengths determined with Equation 3-24 are listed in Table 3-17 in the 2.0 percent row.

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, since this tends to minimize 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 usually begins on the tangent and ends beyond the beginning of 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.

Based on the preceding discussion, locating a portion of the runoff on the tangent is consistent with the natural spiral path adopted by the driver during curve entry. In this manner, the gradual introduction of superelevation prior to the curve compensates for the gradual increase in lateral acceleration associated with the spiral path. As a result, the peak lateral acceleration incurred at the PC should theoretically be equal to 50 percent of the lateral acceleration associated with the circular curve.

To achieve this balance in lateral acceleration, most agencies locate a portion of the runoff length on the tangent prior to the curve. The proportion of runoff length placed on the tangent varies from 0.6 to 0.8

(i.e., 60 to 80 percent) with a large majority of agencies using 0.67 (i.e., 67 percent). Most agencies consistently use a single value of this proportion for all street and highway curves.

Theoretical considerations confirm the desirability of placing a larger portion of the runoff length on the approach tangent rather than on the circular curve. Such considerations are based on analysis of the acceleration acting laterally on the vehicle while it travels through the transition section. This lateral acceleration can induce a lateral velocity and lane shift that could lead to operational problems. Specifically, a lateral velocity in an outward direction (relative to the curve) results in a driver making a corrective steering maneuver that produces a path radius sharper than that of the roadway curve. Such a critical radius produces an undesirable increase in peak side friction demand. Moreover, a lateral velocity of sufficient magnitude to shift the vehicle into an adjacent lane (without corrective steering) is also undesirable for safety reasons.

Analysis of the aforementioned theoretical considerations has led to the conclusion that an appropriate allocation of runoff length between the tangent and the curve can minimize the aforementioned operational problems (*12*). The values obtained from the analysis are listed in Table 3-18. If used in design, the values listed in Table 3-18 should minimize lateral acceleration and the vehicle's lateral motion. Values smaller than those listed tend to be associated with larger outward lateral velocities. Values larger than those listed tend to be associated with larger lateral shifts.

Table 3-18. Runoff Locations that Minimize the Vehicle's Lateral Motion

Metric					U.S. Customary				
Design Speed (km/h)	Portion of Runoff Located prior to the Curve — Number of Lanes Rotated				Design Speed (mph)	Portion of Runoff Located prior to the Curve — Number of Lanes Rotated			
	1.0	1.5	2.0–2.5	3.0–3.5		1.0	1.5	2.0–2.5	3.0–3.5
20–70	0.80	0.85	0.90	0.90	15–45	0.80	0.85	0.90	0.90
80–130	0.70	0.75	0.80	0.85	50–80	0.70	0.75	0.80	0.85

Theoretical considerations indicate that values for the proportion of runoff length on the tangent in the range of 0.7 to 0.9 (i.e., 70 to 90 percent) offer the best operating conditions; the specific value in this range should be dependent on design speed and rotated width. Experience obtained from existing practice indicates that deviation from the values in Table 3-18 by 10 percent should not lead to measurable operational problems. In this regard, use of a single value for the proportion of runoff length on the tangent in the range of 0.6 to 0.9 (60 to 90 percent) for all speeds and rotated widths is considered acceptable. However, refinement of this value, based on the trends shown in Table 3-18 is desirable when conditions allow.

Limiting superelevation rates—Theoretical considerations indicate that, when a vehicle is traveling through a tangent-to-curve transition, large superelevation rates are associated with large shifts in the vehicle's lateral position. In general, such shifts in lateral position can be minimized by the proper location of the superelevation runoff section, as described above. However, large lateral shifts must be compensated by the driver through steering action.

In recognition of the potential adverse effect that large shifts in lateral position may have on vehicle control, the threshold superelevation rates associated with a lateral shift of 1.0 m [3.0 ft] are identified in

Table 3-19. These limiting superelevation rates do not apply for speeds of 80 km/h [50 mph] or more when combined with superelevation rates of 12 percent or less.

Table 3-19. Limiting Superelevation Rates

Metric		U.S. Customary	
Design Speed (km/h)	Limiting Superelevation Rate (%)	Design Speed (mph)	Limiting Superelevation Rate (%)
20	8	15	8
30	8	20	8
40	10	25	10
50	11	30	11
60	11	35	11
70	12	40	11
		45	12

Designs that incorporate superelevation in excess of the limiting rates may be associated with excessive lateral shift. Therefore, it is recommended that such superelevation rates be avoided. However, if they are used, consideration should be given to increasing the width of the traveled way along the curve to reduce the potential for vehicle encroachment into the adjacent lane.

Spiral Curve Transitions

General—Any motor vehicle follows a transition path as it enters or leaves a circular horizontal curve. The steering change and the consequent gain or loss of lateral force cannot be achieved instantly. For most curves, the average driver can follow a suitable transition path within the limits of normal lane width. However, combinations of high speed and sharp curvature lead to longer transition paths, which can result in shifts in lateral position and sometimes actual encroachment on adjoining lanes. In such instances, incorporation of transition curves between the tangent and the sharp circular curve, as well as between circular curves of substantially different radii, may be appropriate to make it easier for a driver to keep the vehicle within its own lane.

The principal advantages of transition curves in horizontal alignment are the following:

1. A properly designed transition curve provides a natural, easy-to-follow path for drivers, such that the lateral force increases and decreases gradually as a vehicle enters and leaves a circular curve. Transition curves minimize encroachment on adjoining traffic lanes and tend to promote uniformity in speed. A spiral transition curve simulates the natural turning path of a vehicle.

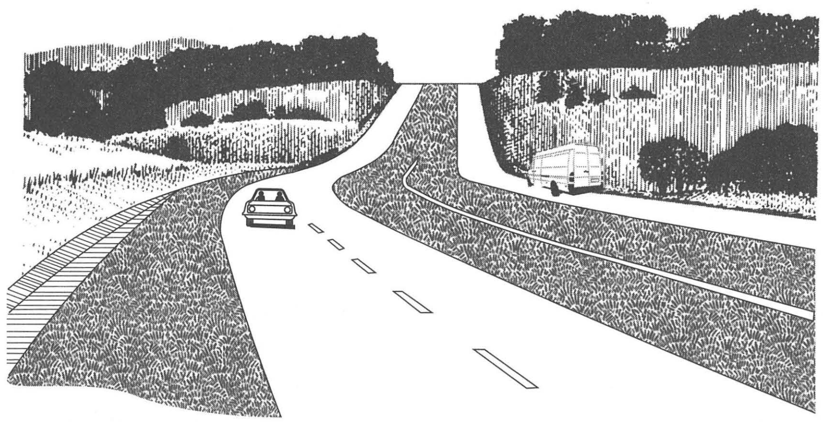
2. The transition curve length provides a suitable location for the superelevation runoff. The transition from the normal pavement cross slope on the tangent to the fully superelevated section on the curve can be accomplished along the length of the transition curve in a manner that closely fits the speed-radius relationship for vehicles traversing the transition. Where superelevation runoff is introduced without a transition curve, usually partly on the curve and partly on the tangent, the driver approaching the curve may need to steer opposite to the direction of the approaching curve when on the superelevated tangent portion to keep the vehicle within its lane.

3. A spiral transition curve also facilitates the transition in width where the traveled way is widened on a circular curve. Use of spiral transitions provides flexibility in accomplishing the widening of sharp curves.

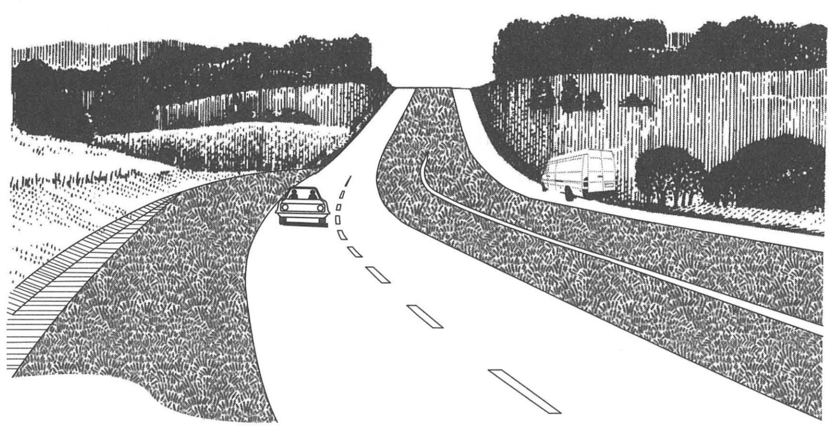
4. The appearance of the highway or street is enhanced by applying spiral transition curves. The use of spiral transitions avoids noticeable breaks in the alignment as perceived by drivers at the beginning and end of circular curves. Figure 3-15 illustrates such breaks, which are more prominent with the presence of superelevation runoff.

Length of Spiral

Length of spiral—Generally, the Euler spiral, which is also known as the clothoid, is used in the design of spiral transition curves. The radius varies from infinity at the tangent end of the spiral to the radius of the circular arc at the end that adjoins that circular arc. By definition, the radius of curvature at any point on an Euler spiral varies inversely with the distance measured along the spiral. In the case of a spiral transition that connects two circular curves having different radii, there is an initial radius rather than an infinite value.



Without Spiral Transition Curves
– A –



With Spiral Transition Curves
– B –

Figure 3-15. Transition Spirals (*63*)

The following equation, developed in 1909 by Shortt (*53*) for gradual attainment of lateral acceleration on railroad track curves, is the basic expression used by some highway agencies for computing minimum length of a spiral transition curve:

Metric	U.S. Customary	
$L = \dfrac{0.0214V^3}{RC}$	$L = \dfrac{3.15V^3}{RC}$	(3-25)
where:	where:	
L = minimum length of spiral, m	L = minimum length of spiral, ft	
V = speed, km/h	V = speed, mph	
R = curve radius, m	R = curve radius, ft	
C = rate of increase of lateral acceleration, m/s^3	C = rate of increase of lateral acceleration, ft/s^3	

The factor C is an empirical value representing the comfort and safety levels provided by the spiral curve. The value of $C = 0.3$ m/s^3 [1 ft/s^3] is generally accepted for railroad operation, but values ranging from 0.3 to 0.9 m/s^3 [1 to 3 ft/s^3] have been used for highways. This equation is sometimes modified to take into account the effect of superelevation, which results in much shorter spiral curve lengths. Highways do not appear to need as much precision as is obtained from computing the length of spiral by this equation or its modified form. A more practical control for the length of spiral is that it should equal the length needed for superelevation runoff.

Maximum radius for use of a spiral—A review of guidance on the use of spiral curve transitions indicates a general lack of consistency among highway agencies. In general, much of this guidance suggests that an upper limit on curve radius can be established such that only radii below this maximum are likely to obtain safety and operational benefits from the use of spiral transition curves. Such a limiting radius has been established by several agencies based on a minimum lateral acceleration rate. Such minimum rates have been found to vary from 0.4 to 1.3 m/s^2 [1.3 to 4.25 ft/s^2]. The upper end of this range of rates corresponds to the maximum curve radius for which some reduction in crash potential has also been noted. For these reasons, it is recommended that the maximum radius for use of a spiral should be based on a minimum lateral acceleration rate of 1.3 m/s^2 [4.25 ft/s^2] (*13*). These radii are listed in Table 3-20.

The radii listed in Table 3-20 are intended for use by those highway agencies that desire to use spiral curve transitions. Table 3-20 is not intended to define radii that need the use of a spiral.

Table 3-20. Maximum Radius for Use of a Spiral Curve Transition

Metric		U.S. Customary	
Design speed (km/h)	Maximum radius (m)	Design speed (mph)	Maximum radius (ft)
20	24	15	114
30	54	20	203
40	95	25	317
50	148	30	456
60	213	35	620
70	290	40	810
80	379	45	1025
90	480	50	1265
100	592	55	1531
110	716	60	1822
120	852	65	2138
130	1000	70	2479
		75	2846
		80	3238

Note: The effect of spiral curve transitions on lateral acceleration is likely to be negligible for larger radii.

Minimum length of spiral—Several agencies define a minimum length of spiral based on consideration of driver comfort and shifts in the lateral position of vehicles. Criteria based on driver comfort are intended to provide a spiral length that allows for a comfortable increase in lateral acceleration as a vehicle enters a curve. The criteria based on lateral shift are intended to provide a spiral curve that is sufficiently long to result in a shift in a vehicle's lateral position within its lane that is consistent with that produced by the vehicle's natural spiral path. It is recommended that these two criteria be used together to determine the minimum length of spiral. Thus, the minimum spiral length can be computed as:

Metric

$L_{s,\min}$ should be the larger of:

$$L_{s,\min} = \sqrt{24(p_{\min})R} \tag{3-26}$$

or

$$L_{s,\min} = 0.0214\frac{V^3}{RC} \tag{3-27}$$

where:

$L_{s,\min}$ = minimum length of spiral, m

$p_{\min}$ = minimum lateral offset between the tangent and circular curve (0.20 m)

R = radius of circular curve, m;

V = design speed, km/h

C = maximum rate of change in lateral acceleration (1.2 m/s^3)

U.S. Customary

$L_{s,\min}$ should be the larger of:

$$L_{s,\min} = \sqrt{24(p_{\min})R} \tag{3-26}$$

or

$$L_{s,\min} = 3.15\frac{V^3}{RC} \tag{3-27}$$

where:

$L_{s,\min}$ = minimum length of spiral, ft

$p_{\min}$ = minimum lateral offset between the tangent and circular curve (0.66 ft)

R = radius of circular curve, ft

V = design speed, mph

C = maximum rate of change in lateral acceleration (4 ft/s^3)

A value of 0.20 m [0.66 ft] is recommended for p_{min}. This value is consistent with the minimum lateral shift that occurs as a result of the natural steering behavior of most drivers. The recommended minimum value for C is 1.2 m/s^3 [4.0 ft/s^3]. The use of lower values will yield longer, more "comfortable" spiral curve lengths; however, such lengths would not represent the minimum length consistent with driver comfort.

Maximum length of spiral—International experience indicates that there is a need to limit the length of spiral transition curves. Spirals should not be so long (relative to the length of the circular curve) that drivers are misled about the sharpness of the approaching curve. A conservative maximum length of spiral that should minimize the likelihood of such concerns can be computed as:

Metric	U.S. Customary
$L_{s,max} = \sqrt{24(p_{max})R}$	$L_{s,max} = \sqrt{24(p_{max})R}$ (3-28)
where:	where:
$L_{s,max}$ = maximum length of spiral, m	$L_{s,max}$ = maximum length of spiral, ft
p_{max} = maximum lateral offset between the tangent and circular curve (1.0 m)	p_{max} = maximum lateral offset between the tangent and circular curve (3.3 ft)
R = radius of circular curve, m	R = radius of circular curve, ft

A value of 1.0 m [3.3 ft] is recommended for p_{max}. This value is consistent with the maximum lateral shift that occurs as a result of the natural steering behavior of most drivers. It also provides a reasonable balance between spiral length and curve radius.

Desirable length of spiral—A study of the operational effects of spiral curve transitions (*13*) found that spiral length is an important design control. Specifically, the most desirable operating conditions were noted when the spiral curve length was approximately equal to the length of the natural spiral path adopted by drivers. Differences between these two lengths resulted in operational problems associated with large lateral velocities or shifts in lateral position at the end of the transition curve. Specifically, a large lateral velocity in an outward direction (relative to the curve) may lead the driver to make a corrective steering maneuver that results in a path radius sharper than the radius of the circular curve. Such a critical radius produces an undesirable increase in peak side friction demand. Moreover, lateral velocities of sufficient magnitude to shift a vehicle into an adjacent lane (without corrective steering) are also undesirable.

Based on these considerations, desirable lengths of spiral transition curves are shown in Table 3-21. These lengths correspond to 2.0 s of travel time at the design speed of the roadway. This travel time has been found to be representative of the natural spiral path for most drivers (*13*).

The spiral lengths listed in Table 3-21 are recommended as desirable values for street and highway design. Theoretical considerations suggest that significant deviations from these lengths tend to increase the shifts in the lateral position of vehicles within a lane that may precipitate encroachment on an adjacent lane or shoulder. The use of longer spiral curve lengths that are less than $L_{s,max}$ is acceptable. However, where such longer spiral curve lengths are used, consideration should be given to increasing the width of the traveled way on the curve to minimize the potential for encroachments into the adjacent lanes.

Spiral curve lengths longer than those shown in Table 3-21 may be needed at turning roadway terminals to adequately develop the desired superelevation. Specifically, spirals twice as long as those shown in Table 3-21 may be needed in such situations. The resulting shift in lateral position may exceed 1.0 m [3.3 ft]; however, such a shift is consistent with driver expectancy at a turning roadway terminal and can be accommodated by the additional lane width typically provided on such turning roadways.

Finally, if the desirable spiral curve length shown in Table 3-21 is less than the minimum spiral curve length determined from Equations 3-26 and 3-27, the minimum spiral curve length should be used in design.

Table 3-21. Desirable Length of Spiral Curve Transition

Metric		U.S. Customary	
Design Speed (km/h)	**Spiral Length (m)**	**Design Speed (mph)**	**Spiral Length (ft)**
20	11	15	44
30	17	20	59
40	22	25	74
50	28	30	88
60	33	35	103
70	39	40	117
80	44	45	132
90	50	50	147
100	56	55	161
110	61	60	176
120	67	65	191
130	72	70	205
		75	220
		80	235

Length of superelevation runoff—In transition design with a spiral curve, it is recommended that the superelevation runoff be accomplished over the length of spiral. For the most part, the calculated values for length of spiral and length of runoff do not differ materially. However, in view of the empirical nature of both, an adjustment in one to avoid having two separate sets of design criteria is desirable. The length of runoff is applicable to all superelevated curves, and it is recommended that this value should be used for minimum lengths of spiral. In this manner, the length of spiral should be set equal to the length of runoff. The change in cross slope begins by introducing a tangent runout section just in advance of the spiral curve. Full attainment of superelevation is then accomplished over the length of the spiral. In such a design, the whole of the circular curve has full superelevation.

Limiting superelevation rates—One consequence of equating runoff length to spiral length is that the resulting relative gradient of the pavement edge may exceed the values listed in Table 3-15. However, small increases in gradient have not been found to have an adverse effect on comfort or appearance. In this regard, the adjustment factors listed in Table 3-16 effectively allow for a 50 percent increase in the maximum relative gradient when three lanes are rotated.

The superelevation rates that are associated with a maximum relative gradient that is 50 percent larger than the values in Table 3-15 are listed in Table 3-22. If the superelevation rate used in design exceeds the rate listed in this table, the maximum relative gradient will be at least 50 percent larger than the maximum relative gradient allowed for a tangent-to-curve design. In this situation, special consideration should be given to the transition's appearance and the abruptness of its edge-of-pavement profile.

Table 3-22. Superelevation Rates Associated with Large Relative Gradients

Metric				U.S. Customary			
Design Speed (km/h)	Number of Lanes Rotated			Design Speed (mph)	Number of Lanes Rotated		
	1	2	3		1	2	3
20	3.7	1.9	1.3	15	4.3	2.2	1.5
30	5.2	2.6	1.7	20	5.5	2.8	1.9
40	6.5	3.2	2.2	25	6.5	3.3	2.2
50	7.5	3.8	2.5	30	7.3	3.7	2.5
60	8.3	4.2	2.8	35	8.0	4.0	2.7
70	8.9	4.5	3.0	40	8.5	4.3	2.9
80	9.3	4.6	3.1	45	8.9	4.5	3.0
90	9.8	4.9	3.3	50	9.2	4.6	3.1
100	10.2	5.1	3.4	55	9.5	4.8	3.2
110	10.4	5.2	3.5	60	9.9	5.0	3.3
120	10.6	5.3	3.5	65	10.3	5.2	3.4
130	10.6	5.3	3.5	70	10.3	5.2	3.5
				75	10.5	5.3	3.5
				80	10.5	5.3	3.5

Note: Based on desirable length of spiral curve transition from Table 3-21.

Length of tangent runout—The tangent runout length for a spiral curve transition design is based on the same approach used for the tangent-to-curve transition design. Specifically, a smooth edge of pavement profile is desired so that a common edge slope gradient is maintained throughout the superelevation runout and runoff sections. Based on this rationale, the following equation can be used to compute the tangent runout length:

Metric	U.S. Customary	
$L_t = \frac{e_{NC}}{e_d} L_S$	$L_t = \frac{e_{NC}}{e_d} L_S$	(3-29)
where:	where:	
L_t = length of tangent runout, m	L_t = length of tangent runout, ft	
L_S = length of spiral, m	L_S = length of spiral, ft	
e_d = design superelevation rate, percent	e_d = design superelevation rate, percent	
e_{NC} = normal cross slope rate, percent	e_{NC} = normal cross slope rate, percent	

The tangent runout lengths obtained from Equation 3-29 are presented in Table 3-23. The lengths in this table may be longer than desirable for combinations of low superelevation rate and high speed. Such long lengths may not provide adequate surface drainage where there is insufficient profile grade. Such concerns can be avoided when the profile grade criteria described in the subsequent portion of this section on "Minimum Transition Grades" are applied to the spiral curve transition.

Table 3-23. Tangent Runout Length for Spiral Curve Transition Design

Metric						U.S. Customary					
Design Speed (km/h)	Tangent Runout Length (m)					Design Speed (mph)	Tangent Runout Length (ft)				
	Superelevation Rate						Superelevation Rate				
	2	4	6	8	10		2	4	6	8	10
20	11	—	—	—	—	15	44	—	—	—	—
30	17	8	—	—	—	20	59	30	—	—	—
40	22	11	7	—	—	25	74	37	25	—	—
50	28	14	9	—	—	30	88	44	29	—	—
60	33	17	11	8	—	35	103	52	34	26	—
70	39	19	13	10	—	40	117	59	39	29	—
80	44	22	15	11	—	45	132	66	44	33	—
90	50	25	17	13	10	50	147	74	49	37	—
100	56	28	19	14	11	55	161	81	54	40	—
110	61	31	20	15	12	60	176	88	59	44	—
120	67	33	22	17	13	65	191	96	64	48	38
130	72	36	24	18	14	70	205	103	68	51	41
						75	220	110	73	55	44
						80	235	118	78	59	47

Notes:

1. Based on 2.0 percent normal cross slope.
2. Superelevation rates above 10 percent and cells with "—" coincide with a pavement edge grade that exceeds the maximum relative gradient in Table 3-15 by 50 percent or more. These limits apply to roads where one lane is rotated; lower limits apply when more lanes are rotated (see Table 3-16).

Location with respect to end of curve—In alignment design with spirals, the superelevation runoff is effected over the whole of the transition curve. The length of the superelevation runoff should be equal to the spiral length for the tangent-to-spiral (TS) transition at the beginning and the spiral-to-curve (SC) transition at the end of the circular curve. The change in cross slope begins by removing the adverse cross slope from the lane or lanes on the outside of the curve on a length of tangent just ahead of TS (the tangent runout) (see Figure 3-16). Between the TS and SC, the spiral curve and the superelevation runoff are coincident and the traveled way is rotated to reach the full superelevation at the SC. This arrangement is reversed on leaving the curve. In this design, the whole of the circular curve has full superelevation.

Compound Curve Transition

In general, compound curve transitions are most commonly considered for application to low-speed turning roadways at intersections. In contrast, tangent-to-curve or spiral curve transition designs are more commonly used on street and highway curves.

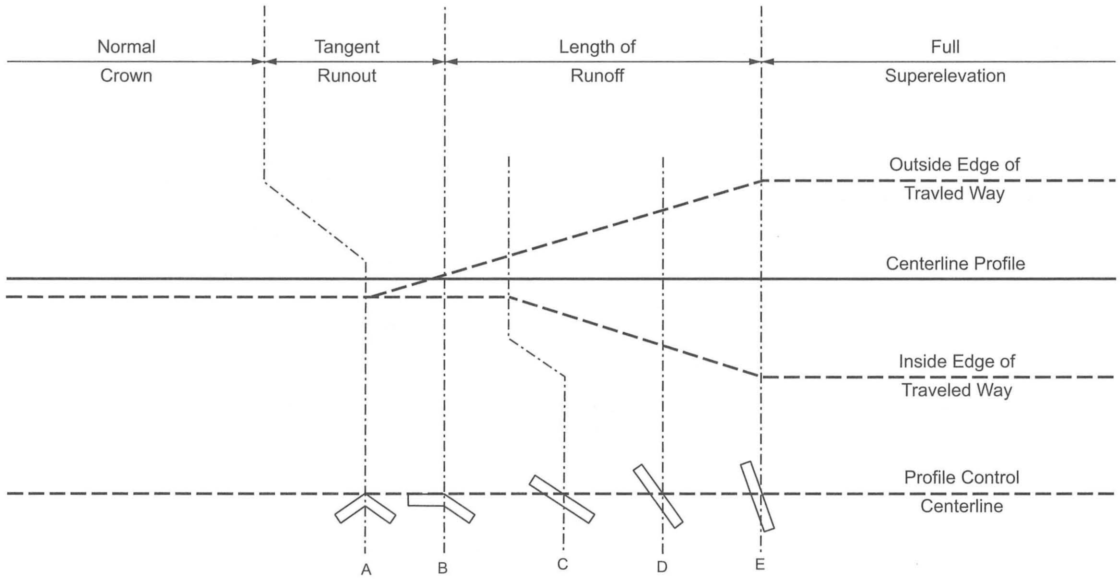
Guidance concerning compound curve transition design for turning roadways is provided in Chapters 9 and 10. The guidance in Chapter 9 applies to low-speed turning roadway terminals at intersections while the guidance in Chapter 10 applies to interchange ramp terminals.

Methods of Attaining Superelevation

Four methods are used to transition the pavement to a superelevated cross section. These methods include: (1) revolving a traveled way with normal cross slopes about the centerline profile, (2) revolving a traveled way with normal cross slopes about the inside-edge profile, (3) revolving a traveled way with normal cross slopes about the outside-edge profile, and (4) revolving a straight cross slope traveled way about the outside-edge profile. Figure 3-16 illustrates these four methods. The methods of changing cross slope are most conveniently shown in the figure in terms of straight line relationships, but it is important that the angular breaks between the straight-line profiles be rounded in the finished design, as shown in the figure.

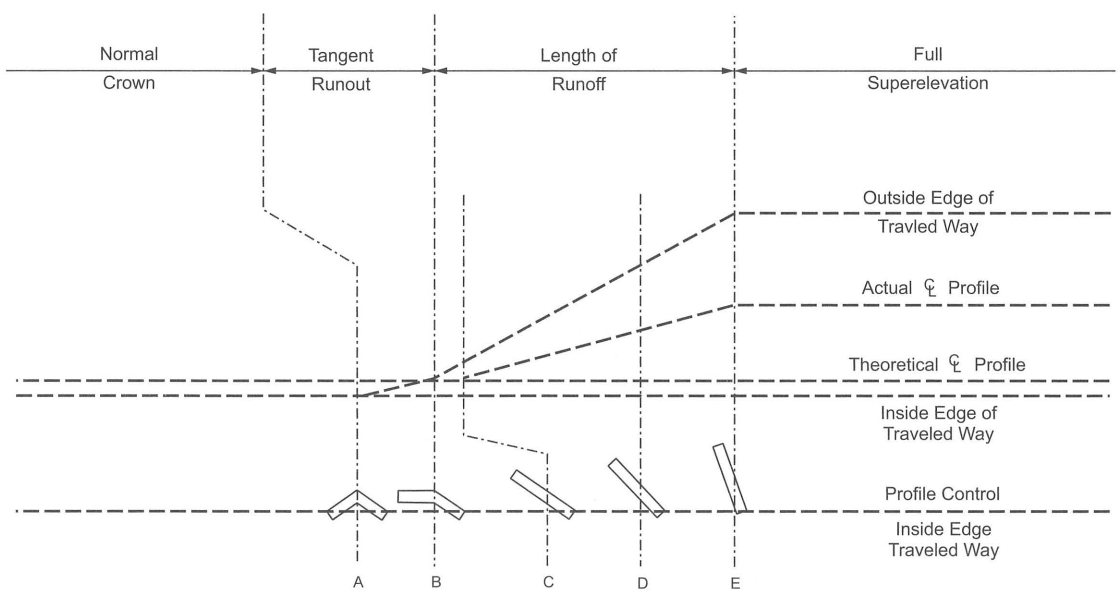
The profile reference line controls for the roadway's vertical alignment through the horizontal curve. Although shown as a horizontal line in Figure 3-16, the profile reference line may correspond to a tangent, a vertical curve, or a combination of the two. In Figure 3-16A, the profile reference line corresponds to the centerline profile. In Figures 3-16B and 3-16C, the profile reference line is represented as a "theoretical" centerline profile as it does not coincide with the axis of rotation. In Figure 3-16D, the profile reference line corresponds to the outside edge of traveled way. The cross sections at the bottom of each diagram in Figure 3-16 indicate the traveled way cross slope condition at the lettered points.

The first method, as shown in Figure 3-16A, revolves the traveled way about the centerline profile. This method is the most widely used because the change in elevation of the edge of the traveled way is achieved with less distortion than with the other methods. In this regard, one-half of the change in elevation is made at each edge.



Crowned
Traveled Way Revolved about Centerline

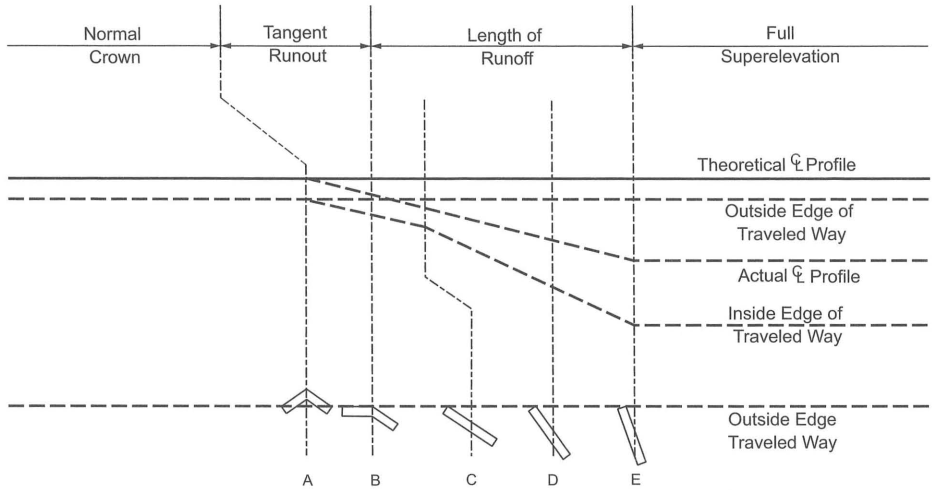
– A –



Crowned
Traveled Way Revolved about Inside Edge

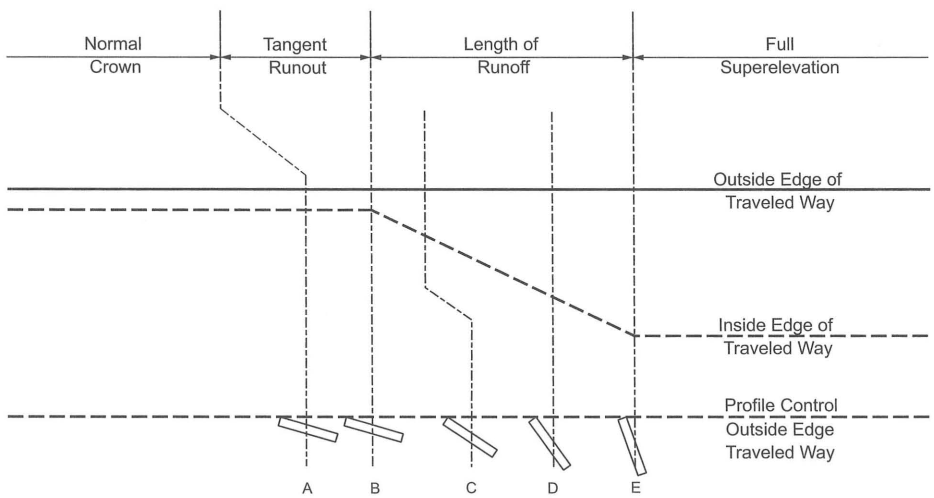
– B –

Figure 3-16. Diagrammatic Profiles Showing Methods of Attaining Superelevation for a Curve to the Right



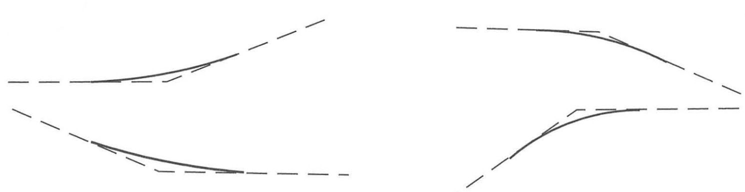
Crowned
Traveled Way Revolved about Outside Edge

– C –



Straight Cross Slope
Traveled Way Revolved about Outside Edge

– D –



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

Figure 3-16. Diagrammatic Profiles Showing Methods of Attaining Superelevation for a Curve to the Right (Continued)

The second method, as shown in Figure 3-16B, 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 3-16C, revolves the traveled way about the outside-edge profile. This method is similar to that shown in Figure 3-16B 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 3-16D, 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 3-16A, 3-16B, and 3-16C; there is no change in cross slope for Figure 3-16D.

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 3-16A, 3-16B, and 3-16C, 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 3-16D. 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 3-16A is usually the most adaptable. On the other hand, the method shown in Figure 3-16B 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 3-16C and 3-16D 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 transition 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.

Design of Smooth Profiles for Traveled-Way Edges

In the diagrammatic profiles shown in Figure 3-16 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. 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 0.65 [0.66] percent break at the 50-km/h [30-mph] design speed (see Figure 3-16) and 0.38 [0.38] percent break at the 120-km/h [75 mph] design speed need. Where the traveled way is revolved about an edge, these grade breaks are doubled to 1.30 [1.32] percent for the 50-km/h [30-mph] design speed and to 0.76 [0.76] percent for the 120-km/h [75-mph] design speed. Greater lengths of vertical curve are obviously needed in these cases. Specific criteria have not been established for the lengths of vertical curves at the breaks in the diagrammatic profiles. For an approximate guide, however, the minimum vertical curve length in meters [feet] can be used as numerically equal to 0.2 times the design speed in kilometers per hour [equal to the design speed in miles 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 3-16. Then by means of a spline, curve template, ship curve, or circular curve, smooth-flowing lines are drawn to approximate the straight-line controls. The natural bending of the spline nearly always satisfies the need for minimum smoothing. Once the edge profiles are drawn in the proper relation to one another, elevations can be read at the appropriate intervals (as needed for construction control).

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

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

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.

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 4 m [15 ft] or less. Superelevation can be attained using a method similar to that shown in Figure 3-16A 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 3 m [10 ft] may be ignored when determining the runoff length.

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 4 and 18 m [15 and 60 ft]. 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 3-16B, 3-16C, and 3-16D, with the profile reference line being the same for both traveled ways. Where Case II is used for a narrow median width of 3 m [10 ft] or less held in a horizontal plane, the runoff lengths may be the same as those for a single undivided highway.

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 18 m [60 ft] 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 3-16 can be used).

Divided highways warrant a greater refinement in design and greater attention to appearance than two-lane highways because they serve much greater traffic volumes and because the cost of such refinements is insignificant compared with the cost of construction. Accordingly, the values for length of runoff previously indicated should be considered minimums, and the use of yet longer values should be considered. Likewise, there should be emphasis on the development of smooth-flowing traveled-way edge profiles of the type obtained by spline-line design methods.

Minimum Transition Grades

Two potential pavement surface drainage problems are of concern in the superelevation transition section. One problem relates to the potential lack of adequate longitudinal grade. This problem generally occurs when the grade axis of rotation is equal, but of opposite sign, to the effective relative gradient. It results in the edge of pavement having negligible longitudinal grade, which can lead to poor pavement surface drainage, especially on curbed cross sections.

The other potential drainage problem relates to inadequate lateral drainage due to negligible cross slope during pavement rotation. This problem occurs in the transition section where the cross slope of the outside lane varies from an adverse slope at the normal cross slope rate to a superelevated slope at the normal cross slope rate. This length of the transition section includes the tangent runout section and an equal length of the runoff section. Within this length, the pavement cross slope may not be sufficient to adequately drain the pavement laterally.

Two techniques can be used to alleviate these two potential drainage problems. One technique is providing a minimum profile grade in the transition section. The second technique is providing a minimum edge-of-pavement grade in the transition section. Both techniques can be incorporated in the design by use of the following grade criteria:

1. Maintain minimum profile grade of 0.5 percent through the transition section.
2. Maintain minimum edge-of-pavement grade of 0.2 percent (0.5 percent for curbed streets) through the transition section.

The second grade criterion is equivalent to the following series of equations relating profile grade and effective maximum relative gradient:

Metric		U.S. Customary	
Uncurbed	Curbed	Uncurbed	Curbed
$G \leq -\Delta^* - 0.2$	$G \leq -\Delta^* - 0.5$	$G \leq -\Delta^* - 0.2$	$G \leq -\Delta^* - 0.5$
$G \geq -\Delta^* + 0.2$	$G \geq -\Delta^* + 0.5$	$G \geq -\Delta^* + 0.2$	$G \geq -\Delta^* + 0.5$
$G \leq \Delta^* - 0.2$	$G \leq \Delta^* - 0.5$	$G \leq \Delta^* - 0.2$	$G \leq \Delta^* - 0.5$
$G \geq \Delta^* + 0.2$	$G \geq \Delta^* + 0.5$	$G \geq \Delta^* + 0.2$	$G \geq \Delta^* + 0.5$

Metric

with

$$\Delta^* = \frac{(wn_l)\, e_d}{L_r} \tag{3-30}$$

where:

G = profile grade, percent

Δ^* = effective maximum relative gradient, percent

w = width of one traffic lane, m (typically 3.6 m)

n_l = number of lanes rotated

e_d = design superelevation rate, percent

L_r = length of superelevation runoff, m

U.S. Customary

with

$$\Delta^* = \frac{(wn_l)\, e_d}{L_r} \tag{3-30}$$

where:

G = profile grade, percent

Δ^* = effective maximum relative gradient, percent

w = width of one traffic lane, ft (typically 12 ft)

n_l = number of lanes rotated

e_d = design superelevation rate, percent

L_r = length of superelevation runoff, ft

The value of 0.2 in the grade control (G) equation represents the minimum edge-of-pavement grade for uncurbed roadways (expressed as a percentage). If this equation is applied to curbed streets, the value 0.2 should be replaced with 0.5.

To illustrate the combined use of the two grade criteria, consider an uncurbed roadway curve having an effective maximum relative gradient of 0.65 percent in the transition section. The first criterion would exclude grades between –0.50 and +0.50 percent. The second grade criterion would exclude grades in the range of –0.85 to –0.45 percent (via the first two components of the equation) and those in the range of 0.45 to 0.85 percent (via the last two components of the equation). Given the overlap between the ranges for Controls 1 and 2, the profile grade within the transition would have to be outside of the range of –0.85 to +0.85 percent to satisfy both criteria and provide adequate pavement surface drainage.

Transitions and Compound Curves for Turning Roadways

Drivers turning at at-grade intersections and at interchange ramp terminals naturally follow transitional travel paths just as they do at higher speeds on the open highway. If facilities are not provided for driving in this natural manner, many drivers may deviate from the intended path and develop their own transition, sometimes to the extent of encroaching on other lanes or on the shoulder. The use of natural travel paths by drivers is best achieved by the use of transition or spiral curves that may be inserted between a tangent and a circular arc or between two circular arcs of different radii. Practical designs that follow transitional paths may also be developed by using compound circular curves. Transitioned roadways have the added advantage of providing a practical means for changing from a normal to a superelevated cross section.

Length of Spiral for Turning Roadways

Lengths of spirals for use at intersections are determined in the same manner as they are for open highways. On intersection curves, lengths of spirals may be shorter than they are on the open highway curves, because drivers accept a more rapid change in direction of travel under intersection conditions. In other words, C (the rate of change of lateral acceleration on intersection curves) may be higher on intersection curves than on open highway curves, where values of C ranging from 0.3 to 1.0 m/s^3 [1 to 3 ft/sec^3] generally are accepted. Rates for curves at intersections are assumed to vary from 0.75 m/s^3 [2.5 ft/s^3] for a turnout speed of 80 km/h [50 mph] to 1.2 m/s^3 [4.0 ft/s^3] for 30 km/h [20 mph]. With the use of these values in the Shortt formula (*53*), lengths of spirals for intersection curves are developed in Table 3-24. The minimum lengths of spirals shown are for minimum-radius curves as governed by the design speed. Somewhat lesser spiral lengths are suitable for above-minimum radii.

Spirals also may be desirable between two circular arcs of widely different radii. In this case, the length of spiral can be obtained from Table 3-24 by using a radius that is the difference in the radii of the two arcs. For example, two curves to be connected by a spiral have radii of 250 and 80 m [820 and 262 ft]. This difference of 170 m [558 ft] is very close to the minimum radius of 160 m [550 ft] in Table 3-24 for which the suggested minimum length is about 60 m [200 ft].

Table 3-24. Minimum Lengths of Spiral for Intersection Curves

Metric					U.S. Customary				
Design Speed (km/h)	Minimum Radius (m)	Assumed C (m/s^3)	Calculated Length of Spiral (m)	Design Minimum Length of Spiral (m)	Design Speed (mph)	Minimum Radius (ft)	Assumed C (ft/s^3)	Calculated Length of Spiral (ft)	Design Minimum Length of Spiral (ft)
30	25	1.2	19	20	20	90	4.0	70	70
40	50	1.1	25	25	25	150	3.75	87	90
50	80	1.0	33	35	30	230	3.5	105	110
60	125	0.9	41	45	35	310	3.25	134	130
70	160	0.8	57	60	40	430	3.0	156	160
					45	550	2.75	190	200

Compound curves at intersections for which the radius of one curve is more than twice the radius of the other should have either a spiral or a circular curve of intermediate radius inserted between the two. If, in such instances, the calculated length of spiral is less than 30 m [100 ft], using a length of at least 30 m [100 ft] is suggested.

Compound Circular Curves

Compound circular curves can effectively create desirable shapes of turning roadways for at-grade intersections and for interchange ramps. Where circular arcs of widely different radii are joined, however, the alignment can appear abrupt or forced, and the travel paths of vehicles will need considerable steering effort.

On compound curves for open highways, it is generally accepted that the ratio of the flatter radius to the sharper radius should not exceed 1.5:1. For compound curves at intersections and on turning roadways where drivers accept more rapid changes in direction and speed, the radius of the flatter arc can be as much as 100 percent greater than the radius of the sharper arc, a ratio of 2:1. The ratio of 2:1 for the sharper curves used at intersections results in approximately the same difference (about 10 km/h [6 mph]) in average running speeds for the two curves. Highway agency experience indicates that ramps having differences in radii with a ratio of 2:1 provide satisfactory operation and appearance for intersections.

Where practical, a smaller difference in radii should be used. A desirable maximum ratio is 1.75:1. Where the ratio is greater than 2:1, a suitable length of spiral or a circular arc of intermediate radius should be inserted between the two curves. In the case of very sharp curves designed to accommodate minimum turning paths of vehicles, it is not practical to apply this ratio control. In this case, compound curves should be developed that fit closely to the path of the design vehicle to be accommodated, for which higher ratios may be needed as shown in Chapter 9.

Curves that are compounded should not be too short or their effectiveness in enabling smooth transitions from tangent or flat-curve to sharp-curve operation may be lost. In a series of curves of decreasing radii, each curve should be long enough to enable the driver to decelerate at a reasonable rate, which at intersections is assumed to be not more than 5 km/h/s [3 mph/s], although 3 km/h/s [2 mph/s] is desirable. Minimum curve lengths that meet these criteria based on the running speeds shown in Table 3-6 are indicated in Table 3-25. They are based on a deceleration of 5 km/h/s [3 mph/s], and a desirable minimum deceleration of 3 km/h/s [2 mph/s]. The latter deceleration rate indicates very light braking, because deceleration in gear alone generally results in deceleration rates between 1.5 and 2.5 km/h/s [1 and 1.5 mph/s].

Table 3-25. Length of Circular Arc for a Compound Intersection Curve When Followed by a Curve of One-Half Radius or Preceded by a Curve of Double Radius

Metric			U.S. Customary		
	Length of Circular Arc (m)			Length of Circular arc (ft)	
Radius (m)	Minimum	Desirable	Radius (ft)	Minimum	Desirable
30	12	20	100	40	60
50	15	20	150	50	70
60	20	30	200	60	90
75	25	35	250	80	120
100	30	45	300	100	140
125	35	55	400	120	180
150 or more	45	60	500 or more	140	200

These design guidelines for compound curves are developed on the premise that travel is in the direction of sharper curvature. For the acceleration condition, the 2:1 ratio is not as critical and may be exceeded.

3.3.9 Offtracking

Offtracking is the characteristic, common to all vehicles, although much more pronounced with the larger design vehicles, in which the rear wheels do not precisely follow the same path as the front wheels when the vehicle traverses a horizontal curve or makes a turn. When a vehicle traverses a curve without superelevation at low speed, the rear wheels track inside the front wheels. When a vehicle traverses a superelevated curve, the rear wheels may track inside the front wheels more or less than they do for a curve without superelevation. This is because of the slip angle of the tires with respect to the direction of travel, which is induced by the side friction developed between the pavement and rolling tires. The relative position of the wheel tracks depends on the speed and the amount of friction developed to sustain the lateral force not offset by superelevation or, when traveling slowly, by the friction developed to counteract the effect of superelevation not compensated by lateral force. At higher speeds, the rear wheels may even track outside the front wheels.

Derivation of Design Values for Widening on Horizontal Curves

In each case, the amount of offtracking, and therefore the amount of widening needed on horizontal curves, depends jointly on the length and other characteristics of the design vehicle and the radius of curvature negotiated. Selection of the design vehicle is based on the size and frequency of the various vehicle types expected at the location in question. The amount of widening that is needed increases with the size of the design vehicle (for single-unit vehicles or vehicles with the same number of trailers or semitrailers) and decreases with the increasing radius of curvature. The width elements of the design vehicle that is used to determine the appropriate roadway widening on curves include: the track width of the design vehicles that may meet or pass on the curve, U; the lateral clearance per vehicle, C; the width of front overhang of the vehicle occupying the inner lane or lanes, F_A; the width of rear overhang, F_B; and a width allowance for the difficulty of driving on curves, Z.

The track width (U) for a vehicle following a curve or making a turn, also known as the swept path width, is the sum of the track width on tangent (u) (2.44 or 2.59 m [8.0 or 8.5 ft] depending on the design vehicle) and the amount of offtracking. The offtracking depends on the radius of the curve or turn, the number and location of articulation points, and the lengths of the wheelbases between axles. The track width on a curve (U) is calculated using the equation:

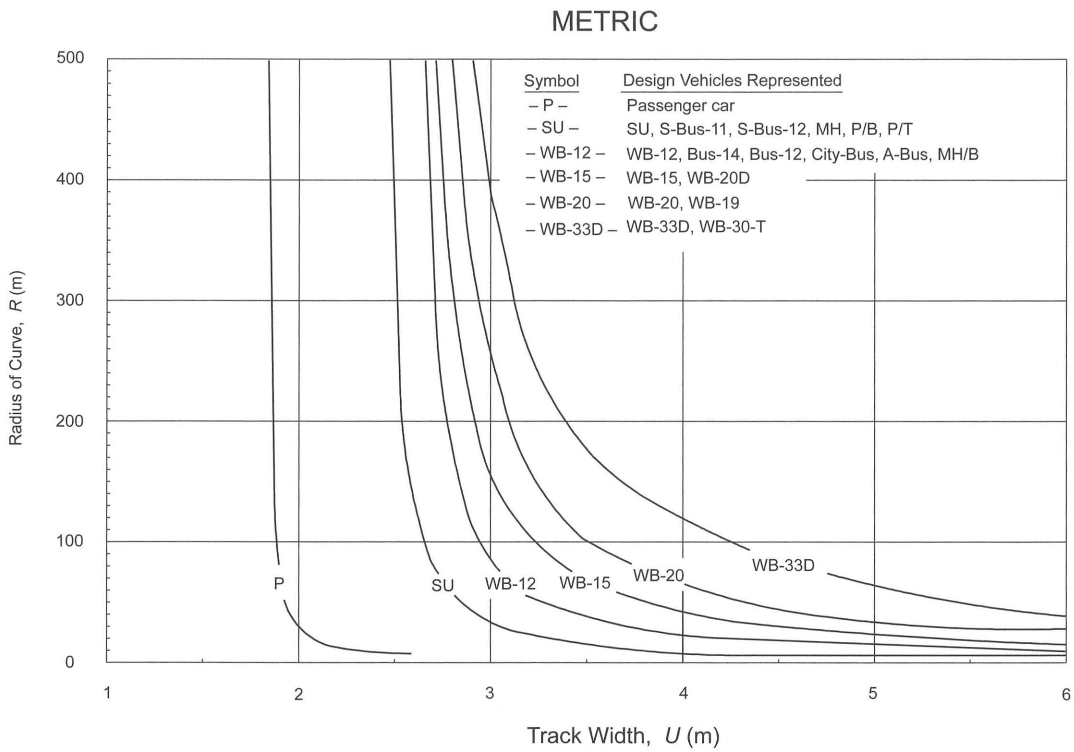
Metric	U.S. Customary	
$U = u + R - \sqrt{R^2 - \sum L_i^2}$	$U = u + R - \sqrt{R^2 - \sum L_i^2}$	(3-31)
where:	where:	
U = track width on curve, m	U = track width on curve, ft	
u = track width on tangent (out-to-out of tires), m	u = track width on tangent (out-to-out of tires), ft	
R = radius of curve or turn, m	R = radius of curve or turn, ft	
L_i = wheelbase of design vehicle between consecutive axles (or sets of tandem axles) and articulation points, m	L_i = wheelbase of design vehicle between consecutive axles (or sets of tandem axles) and articulation points, ft	

This equation can be used for any combination of radius, number of axles, and length of wheelbases (i.e., spacings between axles). The radius for open highway curves is the path of the midpoint of the front axle; however, for most design purposes on two-lane highways, the radius of the curve at the centerline of the highway may be used for simplicity of calculations. For turning roadways, the radius is the path of the outer front wheel (*31*). The wheelbases (L_i) used in the calculations include the distances between each axle and articulation point on the vehicle. For a single-unit truck, only the distance between the front axle and the drive wheels is considered. For an articulated vehicle, each of the articulation points is used to determine U. For example, a tractor/semitrailer combination truck has three L_i values that are considered in determining offtracking: (1) the distance from the front axle to the tractor drive axle(s), (2) the distance from the drive axle(s) to the fifth wheel pivot, and (3) the distance from the fifth wheel pivot to the rear axle(s). In the summation process, some terms may be negative, rather than positive, in two situations: (1) if the articulation point is in front of, rather than behind, the drive axle(s) (*66*) or (2) if there is a rear-axle overhang. Rear-axle overhang is the distance between the rear axle(s) and the pintle hook of a towing vehicle (*31*, *66*) in a multi-trailer combination truck. Representative values for the track width of design vehicles are shown in Figure 3-17 to illustrate the differences in relative widths between groups of design vehicles.

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 0.6, 0.75, and 0.9 m [2.0, 2.5, and 3.0 ft] for tangent lane widths, W_n, equal to 6.0, 6.6, and 7.2 m [20, 22, and 24 ft], 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 3-18 illustrates relative overhang width values for F_A determined from:

Metric	U.S. Customary	
$F_A = \sqrt{R^2 + A(2L + A)} - R$	$F_A = \sqrt{R^2 + A(2L + A)} - R$	(3-32)
where:	where:	
F_A = width of front overhang, m	F_A = width of front overhang, ft	
R = radius of curve or turning roadway (two-lane), m	R = radius of curve or turning roadway (two-lane), ft	
A = front overhang of inner lane vehicle, m	A = front overhang of inner lane vehicle, ft	
L = wheelbase of single unit or tractor, m	L = wheelbase of single unit or tractor, ft	



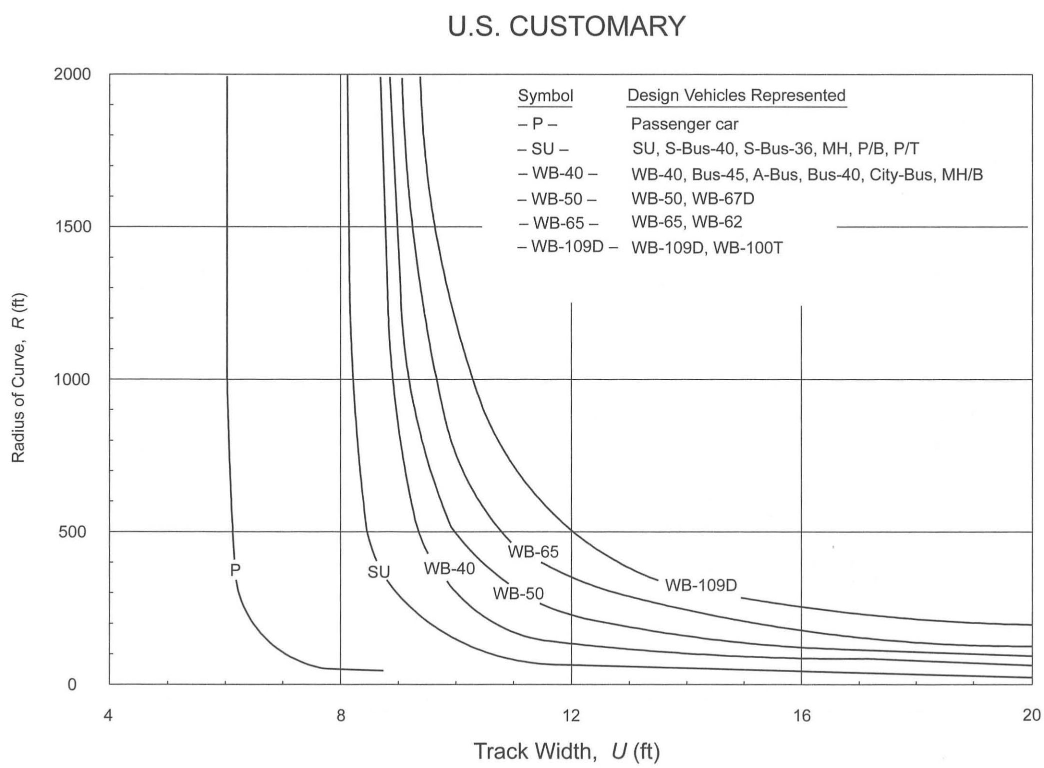
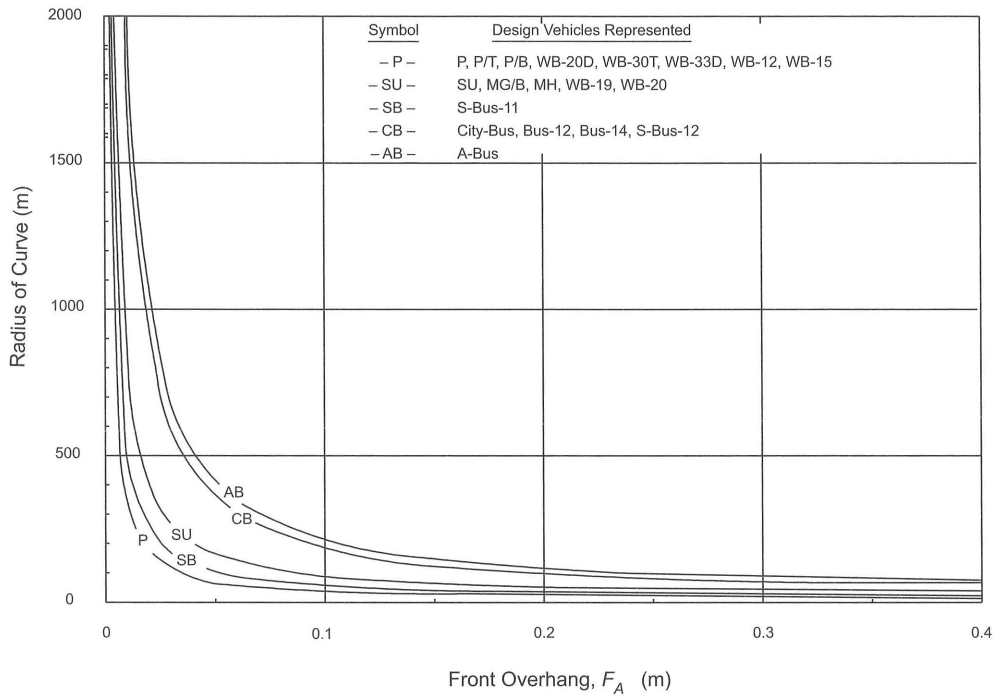


Figure 3-17. Track Width for Widening of Traveled Way on Curves

METRIC



U.S. CUSTOMARY

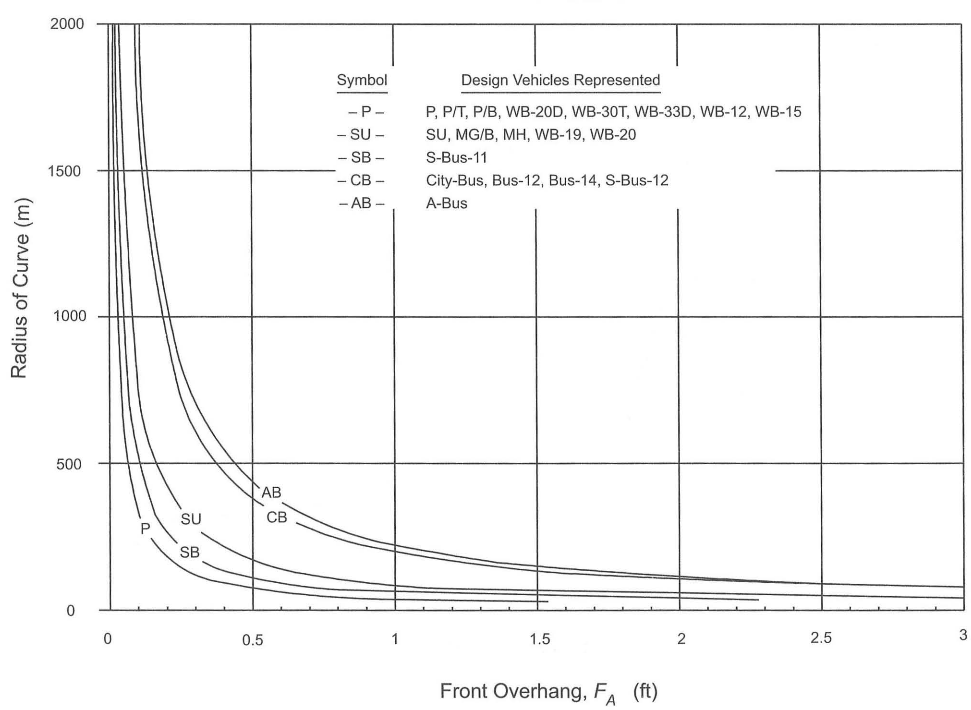


Figure 3-18. Front Overhang for Widening of Traveled Way on Curves

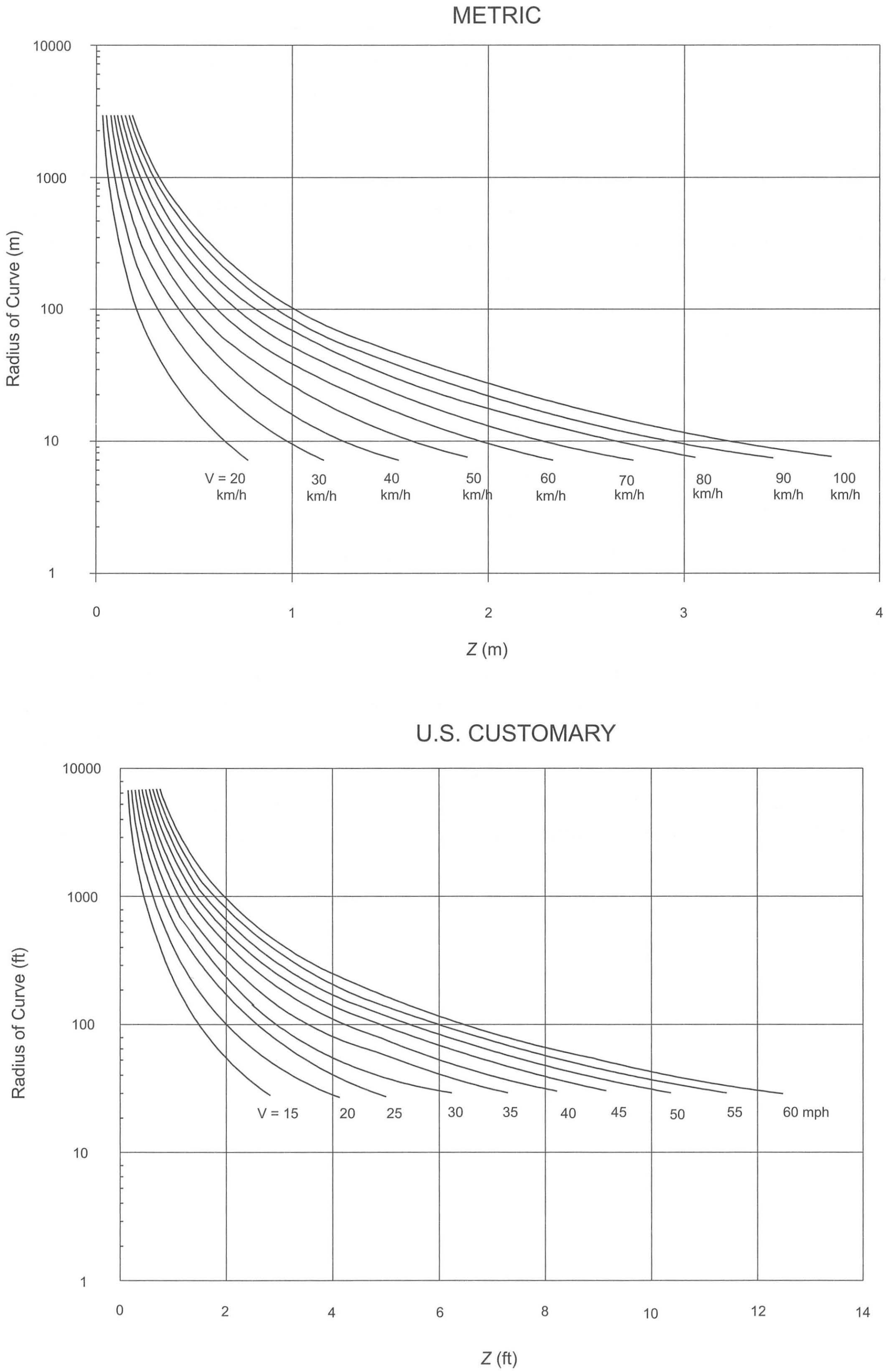


Figure 3-19. Extra Width Allowance for Difficulty of Driving on Traveled Way on Curves

The width of the rear overhang (F_B) 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 0.3 m [1 ft] greater than the width of out-to-out width of the rear wheels, making $F_B = 0.15$ m [0.5 ft]. In the truck design vehicles, the width of body is the same as the width out-to-out of the rear wheels, and $F_B = 0$.

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:

Metric	U.S. Customary	
$Z = 0.1\left(V/\sqrt{R}\right)$	$Z = V/\sqrt{R}$	(3-33)
where: Z = extra width allowance, m V = design speed of the highway, km/h R = radius of curve or turning roadway (two-lane), m	where: Z = extra width allowance, ft V = design speed of the highway, mph R = radius of curve or turning roadway (two-lane), ft	

This expression, used primarily for widening of the traveled way on open highways, is also applicable to intersection curves. Figure 3-19 illustrates the computed values for Z for speeds between 20 and 100 km/h [15 and 60 mph]. For the normal range of curve radii at intersections, Z converges to a nearly constant value of 0.6 m [2 ft] by using the speed-curvature relations for radii in the range of 15 to 150 m [50 to 500 ft]. This added width, as shown diagrammatically in Figures 3-20 and 3-21, should be assumed to be evenly distributed over the traveled way width to allow for the inaccuracy in steering on curved paths.

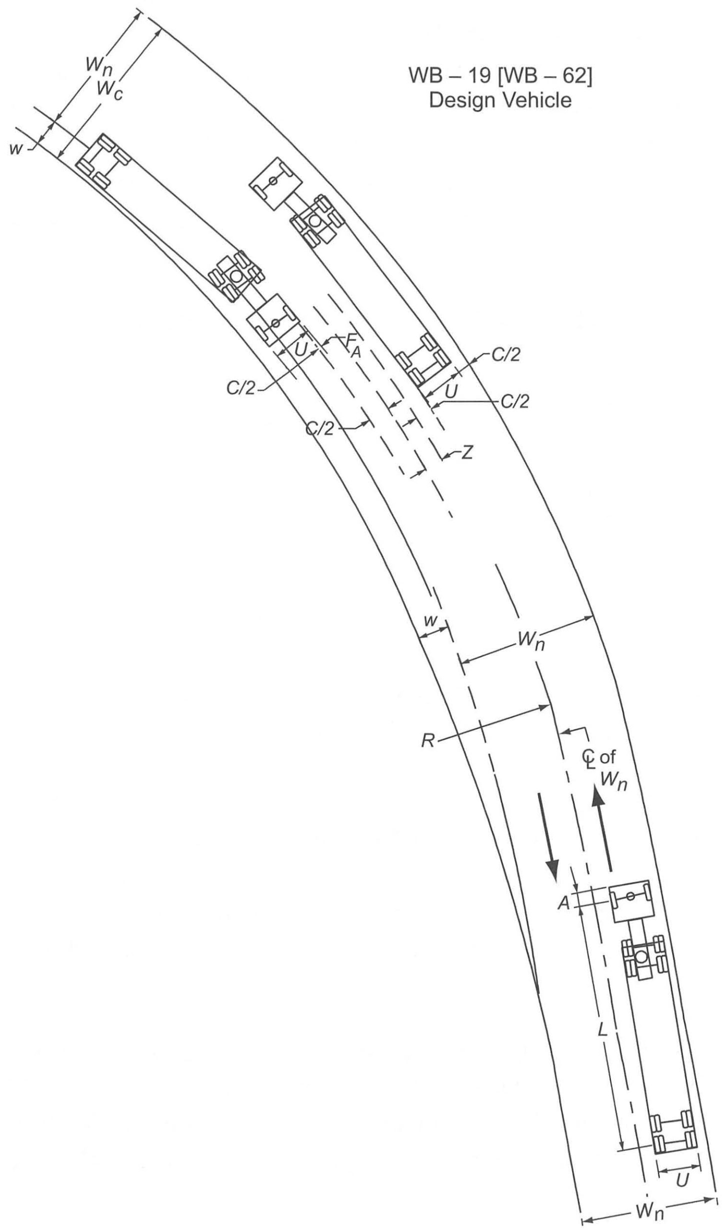


Figure 3-20. Widening Components on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)

3.3.10 Traveled-Way Widening on Horizontal Curves

The traveled way on horizontal curves is sometimes widened to create operating conditions on curves that are comparable to those on tangents. On earlier highways with narrow lanes and sharp curves, there was considerable need for widening on curves, even though speeds were generally low. On modern highways and streets with 3.6-m [12-ft] lanes and high-type alignment, the need for widening has lessened considerably in spite of high speeds, but for some conditions of speed, curvature, and width, it remains appropriate to widen traveled ways.

Widening is needed on certain curves for one of the following reasons: (1) the design vehicle occupies a greater width because the rear wheels generally track inside front wheels (offtracking) in negotiating curves, or (2) drivers experience difficulty in steering their vehicles in the center of the lane. The added width occupied by the vehicle as it traverses the curve as compared with the width of the traveled way on tangent can be computed by geometry for any combination of radius and wheelbase. The effect of variation in lateral placement of the rear wheels with respect to the front wheels and the resultant difficulty of steering should be accommodated by widening on curves, but the appropriate amount of widening cannot be determined as precisely as that for simple offtracking.

The amount of widening of the traveled way on a horizontal curve is the difference between the width needed on the curve and the width used on a tangent:

Metric	U.S. Customary	
$w = W_c - W_n$	$w = W_c - W_n$	(3-34)
where:	where:	
w = widening of traveled way on curve, m	w = widening of traveled way on curve, ft	
W_c = width of traveled way on curve, m	W_c = width of traveled way on curve, ft	
W_n = width of traveled way on tangent, m	W_n = width of traveled way on tangent, ft	

The traveled-way width needed on a curve, W_c, has several components related to operation on curves, including the track width of each vehicle meeting or passing, U; the lateral clearance for each vehicle, C; width of front overhang of the vehicle occupying the inner lane or lanes, F_A; and a width allowance for the difficulty of driving on curves, Z. The application of these components is illustrated in Figure 3-20. Each of these components is derived in Section 3.3.9 under "Derivation of Design Values for Widening on Horizontal Curves."

To determine width W_c, it is necessary to select an appropriate design vehicle. The design vehicle should usually be a truck because offtracking is much greater for trucks than for passenger cars. The WB-19 [WB-62] design vehicle is considered representative for two-lane open-highway conditions. However, other design vehicles may be selected when they better represent the larger vehicles in the actual traffic on a particular facility.

The traveled-way widening values for the assumed design condition for a WB-19 [WB-62] vehicle on a two-lane highway are presented in Table 3-26. The differences in track widths of the SU, WB-12, WB-19, WB-20, WB-20D, WB-30T, and WB-33D [SU, WB-40, WB-62, WB 65, WB-67D, WB-100T, and WB-109D] design trucks are substantial for the sharp curves associated with intersections, but for open highways on which radii are usually larger than 200 m [650 ft], with design speeds over 50 km/h [30 mph], the differences are insignificant (see Figure 3-17). Where both sharper curves (as for a 50 km/h [30 mph] design speed) and large truck combinations are prevalent, the derived widening values for the WB-19 [WB-62] truck should be adjusted in accordance with Table 3-27. 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, as shown in Figure 3-19 for the shorter radii. With the lower speeds and volumes on roads with such curvature, however, slightly smaller clearances may be appropriate.

Table 3-26a. Calculated and Design Values For Traveled Way Widening on Open Highway Curves (Two-Lane Highways, One-Way Or Two-Way)

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

Notes:

Values shown are for WB-19 design vehicle and represent widening in meters. For other design vehicles, use adjustments in Table 3-27.

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.

Table 3-26b. Calculated and Design Values for Traveled Way Widening on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)

U.S. Customary																						
Radius of Curve (ft)	Roadway width = 24 ft							Roadway width = 22 ft							Roadway width = 20 ft							
	Design Speed (mph)							Design Speed (mph)							Design Speed (mph)							
	30	35	40	45	50	55	60	30	35	40	45	50	55	60	30	35	40	45	50	55	60
7000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.7	0.7	0.8	0.8	0.9	1.0	1.0	1.7	1.7	1.8	1.8	1.9	2.0	2.0
6500	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.7	1.8	1.8	1.9	2.0	2.0	2.1
6000	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.7	1.8	1.9	1.9	2.0	2.1	2.1
5500	0.0	0.0	0.0	0.0	0.1	0.1	0.2	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.8	1.9	1.9	2.0	2.1	2.1	2.2
5000	0.0	0.0	0.0	0.1	0.1	0.2	0.3	0.9	0.9	1.0	1.1	1.1	1.2	1.3	1.9	1.9	2.0	2.1	2.1	2.2	2.3
4500	0.0	0.0	0.1	0.1	0.2	0.3	0.4	0.9	1.0	1.1	1.1	1.2	1.3	1.4	1.9	2.0	2.1	2.1	2.2	2.3	2.4
4000	0.0	0.1	0.2	0.2	0.3	0.4	0.5	1.0	1.1	1.2	1.2	1.3	1.4	1.5	2.0	2.1	2.2	2.2	2.3	2.4	2.5
3500	0.1	0.2	0.3	0.4	0.5	0.5	0.6	1.1	1.2	1.3	1.4	1.5	1.5	1.6	2.1	2.2	2.3	2.4	2.5	2.5	2.6
3000	0.3	0.4	0.4	0.5	0.6	0.7	0.8	1.3	1.4	1.4	1.5	1.6	1.7	1.8	2.3	2.4	2.4	2.5	2.6	2.7	2.8
2500	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.5	2.6	2.7	2.8	2.9	3.0	3.1
2000	0.7	0.9	1.0	1.1	1.2	1.3	1.4	1.7	1.9	2.0	2.1	2.2	2.3	2.4	2.7	2.9	3.0	3.1	3.2	3.3	3.4
1800	0.9	1.0	1.1	1.3	1.4	1.5	1.6	1.9	2.0	2.1	2.3	2.4	2.5	2.6	2.9	3.0	3.1	3.3	3.4	3.5	3.6
1600	1.1	1.2	1.3	1.5	1.6	1.7	1.8	2.1	2.2	2.3	2.5	2.6	2.7	2.8	3.1	3.2	3.3	3.5	3.6	3.7	3.8
1400	1.3	1.5	1.6	1.7	1.9	2.0	2.1	2.3	2.5	2.6	2.7	2.9	3.0	3.1	3.3	3.5	3.6	3.7	3.9	4.0	4.1
1200	1.7	1.8	1.9	2.1	2.2	2.4	2.5	2.7	2.8	2.9	3.1	3.2	3.4	3.5	3.7	3.8	3.9	4.1	4.2	4.4	4.5
1000	2.1	2.3	2.4	2.6	2.7	2.9	3.0	3.1	3.3	3.4	3.6	3.7	3.9	4.0	4.1	4.3	4.4	4.6	4.7	4.9	5.0
900	2.4	2.6	2.7	2.9	3.1	3.2		3.4	3.6	3.7	3.9	4.1	4.2		4.4	4.6	4.7	4.9	5.1	5.2	
800	2.7	2.9	3.1	3.3	3.5	3.6		3.7	3.9	4.1	4.3	4.5	4.6		4.7	4.9	5.1	5.3	5.5	5.6	
700	3.2	3.4	3.6	3.8	4.0			4.2	4.4	4.6	4.8	5.0			5.2	5.4	5.6	5.8	6.0		
600	3.8	4.0	4.2	4.4	4.6			4.8	5.0	5.2	5.4	5.6			5.8	6.0	6.2	6.4	6.6		
500	4.6	4.9	5.1	5.3				5.6	5.9	6.1	6.3				6.6	6.9	7.1	7.3			
450	5.2	5.4	5.7					6.2	6.4	6.7					7.2	7.4	7.7				
400	5.9	6.1	6.4					6.9	7.1	7.4					7.9	8.1	8.4				
350	6.8	7.0	7.3					7.8	8.0	8.3					8.8	9.0	9.3				
300	7.9	8.2						8.9	9.2						9.9	10.2					
250	9.6							10.6							11.6						
200	12.0							13.0							14.0						

Notes:

Values shown are for WB-19 design vehicle and represent widening in feet. For other design vehicles, use adjustments in Table 3-27.

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.

Design Values for Traveled-Way Widening

Widening is costly and very little is actually gained from a small amount of widening. It is suggested that a minimum widening of 0.6 m [2.0 ft] be used and that lower values in Table 3-26 be disregarded. Note that the values in Table 3-26 are for a WB-19 [WB-62] design vehicle. For other design vehicles, an adjustment from Table 3-27 should be applied. Values in Table 3-26 also are applicable to two-lane, one-way traveled ways (i.e., to each roadway of a divided highway or street). Studies show that on tangent alignment somewhat smaller clearances between vehicles are used in passing vehicles traveling in the same direction as compared with meeting vehicles traveling in opposite directions. There is no evidence that these smaller clearances are obtained on curved alignment on one-way roads. Moreover, drivers are not able to judge clearances as well when passing vehicles as when meeting opposing vehicles on a curved two-way highway. For this reason and because all geometric elements on a divided highway are generally well maintained, widening on a two-lane, one-way traveled way of a divided highway should be the same as that on a two-lane, two-way highway, as noted in Table 3-26.

Application of Widening on Curves

Widening should transition gradually on the approaches to the curve to provide a reasonably smooth alignment of the edge of the traveled way and to fit the paths of vehicles entering or leaving the curve. The principal points of concern in the design of curve widening, which apply to both ends of highway curves, are presented as follows:

- On simple (unspiraled) curves, widening should be applied on the inside edge of the traveled way only. On curves designed with spirals, widening may be applied on the inside edge or divided equally on either side of the centerline. In the latter method, extension of the outer-edge tangent avoids a slight reverse curve on the outer edge. In either case, the final marked centerline, and preferably any central longitudinal joint, should be placed midway between the edges of the widened traveled way.

- Curve widening should transition gradually over a length sufficient to make the whole traveled way fully usable. Although a long transition is desirable for traffic operation, it may result in narrow pavement slivers that are difficult and expensive to construct. Preferably, widening should transition over the superelevation runoff length, but shorter lengths are sometimes used. Changes in width normally should be effected over a distance of 30 to 60 m [100 to 200 ft].

Table 3-27. Adjustments for Traveled Way Widening Values on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way)

Metric									U.S. Customary								
Radius of Curve (m)	Design Vehicle								Radius of Curve (ft)	Design Vehicle							
	SU-9	SU-12	WB-12	WB-20	WB-20D	WB-28D	WB-30T	WB-33D		SU-30	SU-40	WB-40	WB-67	WB-67D	WB-92D	WB-100T	WB-109D
3000	−0.4	−0.3	−0.3	0.0	0.0	0.0	0.0	0.0	7000	−1.2	−1.2	−1.2	0.1	−0.1	0.1	−0.1	0.2
25000	−0.4	−0.4	−0.3	0.0	0.0	0.0	0.0	0.1	6500	−1.3	−1.2	−1.2	0.1	−0.1	0.1	−0.1	0.2
2000	−0.4	−0.4	−0.4	0.0	0.0	0.0	0.0	0.1	6000	−1.3	−1.2	−1.2	0.1	−0.1	0.1	−0.1	0.2
1500	−0.4	−0.4	−0.4	0.0	−0.1	0.0	0.0	0.1	5500	−1.3	−1.3	−1.2	0.1	−0.2	0.1	−0.1	0.2
1000	−0.5	−0.4	−0.4	0.0	−0.1	0.0	0.0	0.1	5000	−1.3	−1.3	−1.3	0.1	−0.2	0.1	−0.1	0.3
900	−0.5	−0.4	−0.4	0.0	−0.1	0.0	0.0	0.1	4500	−1.4	−1.3	−1.3	0.1	−0.2	0.1	−0.1	0.3
800	−0.5	−0.5	−0.4	0.0	−0.1	0.0	0.0	0.2	4000	−1.4	−1.4	−1.3	0.1	−0.2	0.1	−0.1	0.3
700	−0.5	−0.5	−0.5	0.1	−0.1	0.1	0.0	0.2	3500	−1.5	−1.4	−1.4	0.1	−0.3	0.1	−0.1	0.4
600	−0.6	−0.5	−0.5	0.1	−0.1	0.1	−0.1	0.2	3000	−1.6	−1.5	−1.4	0.1	−0.3	0.1	−0.1	0.5
500	−0.6	−0.6	−0.5	0.1	−0.2	0.1	−0.1	0.3	2500	−1.7	−1.6	−1.5	0.2	−0.4	0.2	−0.1	0.5
400	−0.7	−0.6	−0.6	0.1	−0.2	0.1	−0.1	0.3	2000	−1.8	−1.7	−1.6	0.2	−0.5	0.2	−0.2	0.7
300	−0.8	−0.7	−0.7	0.1	−0.3	0.1	−0.1	0.4	1800	−1.9	−1.8	−1.7	0.2	−0.5	0.2	−0.2	0.8
250	−0.9	−0.8	−0.8	0.1	−0.3	0.2	−0.1	0.5	1600	−2.0	−1.9	−1.8	0.2	−0.6	0.3	−0.2	0.8
200	−1.1	−1.0	−0.9	0.2	−0.4	0.2	−0.2	0.6	1400	−2.2	−2.0	−1.9	0.3	−0.6	0.3	−0.3	1.0
150	−1.3	−1.2	−1.1	0.2	−0.6	0.3	−0.2	0.8	1200	−2.4	−2.2	−2.1	0.3	−0.8	0.3	−0.3	1.1
140	−1.4	−1.2	−1.2	0.3	−0.6	0.3	−0.2	0.9	1000	−2.7	−2.4	−2.3	0.4	−0.9	0.4	−0.4	1.4
130	−1.5	−1.3	−1.2	0.3	−0.6	0.3	−0.2	1.0	900	−2.8	−2.6	−2.4	0.4	−1.0	0.5	−0.4	1.5
120	−1.6	−1.4	−1.3	0.3	−0.7	0.3	−0.3	1.1	800	−3.1	−2.8	−2.6	0.5	−1.1	0.5	−0.4	1.7
110	−1.7	−1.5	−1.4	0.3	−0.8	0.4	−0.3	1.2	700	−3.4	−3.0	−2.9	0.6	−1.3	0.6	−0.5	1.9
100	−1.8	−1.6	−1.5	0.4	−0.8	0.4	−0.3	1.3	600	−3.8	−3.4	−3.2	0.7	−1.5	0.7	−0.6	2.3
90	−2.0	−1.8	−1.6	0.4	−0.9	0.4	−0.4	1.4	500	−4.3	−3.8	−3.6	0.8	−1.8	0.8	−0.7	2.7
80	−2.2	−1.9	−1.8	0.5	−1.0	0.5	−0.4	1.6	450	−4.7	−4.2	−3.9	0.9	−2.0	0.9	−0.8	3.0
70	−2.5	−2.2	−2.0	0.5	−1.2	0.6	−0.5	1.9	400	−5.2	−4.6	−4.3	1.0	−2.3	1.0	−0.9	3.4
									350	−5.8	−5.1	−4.7	1.1	−2.6	1.2	−1.0	3.9
									300	−6.6	−5.8	−5.4	1.3	−3.0	1.4	−1.2	4.6
									250	−7.7	−6.7	−6.3	1.6	−3.6	1.7	−1.4	5.5
									200	−9.4	−8.2	−7.6	2.0	−4.6	2.1	−1.8	7.0

Notes:

Adjustments are applied by adding to or subtracting from the values in Table 3-26.

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

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

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

The width W_c is calculated by the equation:

Metric	U.S. Customary
$W_c = N(U+C)+(N-1)F_A+Z$	$W_c = N(U+C)+(N-1)F_A+Z$ (3-35)
where:	where:
W_c = width of traveled way on curve, m	W_c = width of traveled way on curve, ft
N = number of lanes	N = number of lanes
U = track width of design vehicle (out-to-out tires) on curves, m	U = track width of design vehicle (out-to-out tires) on curves, ft
C = lateral clearance, m	C = lateral clearance, ft
F_A = width of front overhang of inner-lane vehicle, m	F_A = width of front overhang of inner-lane vehicle, ft
Z = extra width allowance, m	Z = extra width allowance, ft

- From the standpoints of usefulness and appearance, the edge of the traveled way through the widening transition should be a smooth, graceful curve. A tangent transition edge should be avoided. On minor highways or in cases where plan details are not available, a curved transition staked by eye generally is satisfactory and better than a tangent transition. In any event, the transition ends should avoid an angular break at the pavement edge.

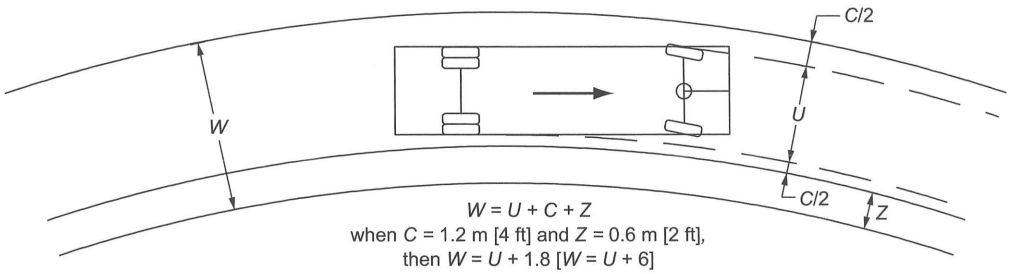
- On highway alignment without spirals, smooth and fitting alignment results from attaining widening with one-half to two-thirds of the transition length along the tangent and the balance along the curve. This is consistent with a common method for attaining superelevation. The inside edge of the traveled way may be designed as a modified spiral, with control points determined by either the width/length ratio of a triangular wedge, by calculated values based on a parabolic or cubic curve, or by a larger radius (compound) curve. Otherwise, it may be aligned by eye in the field. On highway alignment with spiral curves, the increase in width is usually distributed along the length of the spiral.

- Widening areas can be fully detailed on construction plans. Alternatively, general controls can be cited on construction or standard plans with final details left to the field engineer.

3.3.11 Widths for Turning Roadways at Intersections

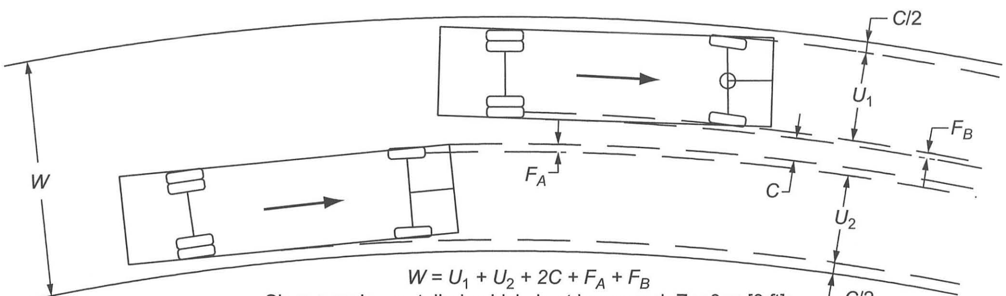
The widths of turning roadways at intersections are governed by the types of vehicles to be accommodated, the radius of curvature, and the expected speed. Turning roadways may be designed for one- or two-way operation, depending on the geometric pattern of the intersection.

Selection of an appropriate design vehicle should be based on the size and frequency of vehicle types using or expected to use the facility. The radius of curvature in combination with the track width of the design vehicle determine the width of a turning roadway. The width elements for the turning vehicle, shown diagrammatically in Figure 3-21, are explained in "Derivation of Design Values for Widening on Horizontal Curves" of Section 3.3.9. They ignore the effects of insufficient superelevation and of surfaces with low friction that tend to cause the rear wheels of vehicles traveling at other than low speed to swing outward, developing the appropriate slip angles.



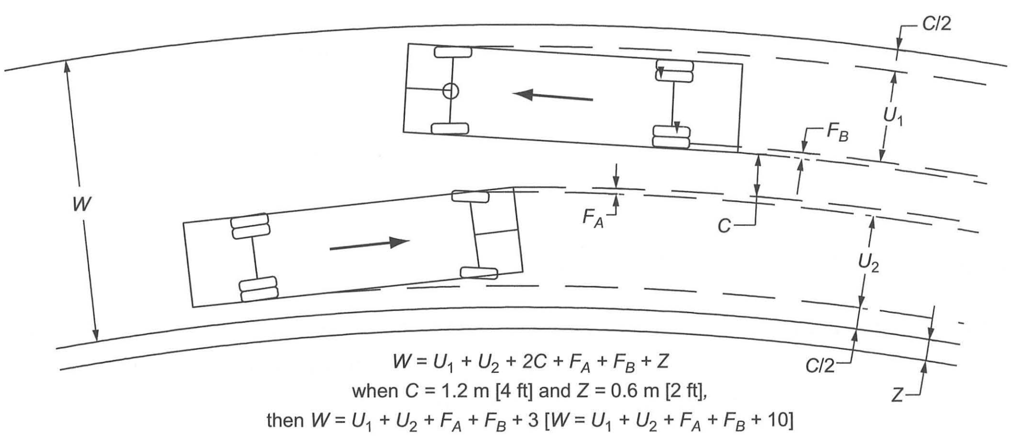
$W = U + C + Z$
when C = 1.2 m [4 ft] and Z = 0.6 m [2 ft],
then $W = U + 1.8$ [$W = U + 6$]

CASE I
One-Lane One-Way Operation — No Passing



$W = U_1 + U_2 + 2C + F_A + F_B$
Since passing a stalled vehicle is at low speed, Z = 0 m [0 ft];
then C is assumed half that for Cases I and III or C = 0.6 m [2 ft],
then $W = U_1 + U_2 + F_A + F_B + 1.2$ [$W = U_1 + U_2 + F_A + F_B + 4$]

CASE II
One-Lane One-Way Operation Provision for Passing Stalled Vehicle



$W = U_1 + U_2 + 2C + F_A + F_B + Z$
when C = 1.2 m [4 ft] and Z = 0.6 m [2 ft],
then $W = U_1 + U_2 + F_A + F_B + 3$ [$W = U_1 + U_2 + F_A + F_B + 10$]

CASE III
Two-Lane Operation — One or Two Way

U = Track Width of Vehicle (Out-to-Out Tires), m [ft]
F_A = Width of Front Overhang, m [ft]
F_B = Width of Rear Overhang, m [ft]
C = Total Lateral Clearance per Vehicle, m [ft]
Z = Extra Width Allowance Due to Difficulty of Driving on Curves, m [ft]

Figure 3-21. Derivation of Turning Roadway Widths on Curves at Intersections

Turning roadways are classified for operational purposes as one-lane operation, with or without opportunity for passing a stalled vehicle, and two-lane operation, either one-way or two-way. Three cases are commonly considered in design:

Case I—One-lane, one-way operation with no provision for passing a stalled vehicle is usually appropriate for minor turning movements and moderate turning volumes where the connecting roadway is relatively short. Under these conditions, the chance of a vehicle breakdown is remote, but one of the edges of the traveled way should preferably have a sloping curb or be flush with the shoulder.

Case II—One-lane, one-way operation with provision for passing a stalled vehicle is used to allow operation at low speed and with sufficient clearance so that other vehicles can pass a stalled vehicle. These widths are applicable to all turning movements of moderate to heavy traffic volumes that do not exceed the capacity of a single-lane connection. In the event of a breakdown, traffic flow can be maintained at a somewhat reduced speed. Many ramps and connections at channelized intersections are in this category. However, for Case II, the widths needed for the longer vehicles are very large as shown in Table 3-28. Case I widths for these longer vehicles, including the WB-19, WB-20, WB-30T, and WB-33D [WB-62, WB-65, WB-100T, and WB-109D] design vehicles, may have to be used as the minimum values where they are present in sufficient numbers to be considered the appropriate design vehicle.

Case III—Two-lane operation, either one- or two-way, is applicable where operation is two way or where operation is one way, but two lanes are needed to handle the traffic volume.

Table 3-28a. Derived Pavement Widths for Turning Roadways for Different Design Vehicles

Metric																				
Radius on Inner Edge of Pavement, *R* (m)	Case I, One-Lane Operation, No Provision for Passing a Stalled Vehicle																			
	P	SU-9	SU-12	BUS-12	BUS-14	CITY-BUS	S-BUS-11	S-BUS-12	A-BUS-11	WB-12	WB-19	WB-20	WB-20D	WB-28D	WB-30T	WB-33D	MH	P/T	P/B	MH/B
15	4.0	5.5	6.3	6.6	7.2	6.5	5.7	5.5	6.7	7.0	13.5	—	8.8	—	11.6	—	5.5	5.7	5.4	6.5
25	3.9	5.0	5.4	5.7	5.9	5.6	5.1	5.0	5.7	5.8	8.5	9.5	6.8	9.6	7.9	12.0	5.0	5.1	4.9	5.5
30	3.8	4.9	5.2	5.4	5.7	5.4	5.0	4.9	5.5	5.5	7.8	8.5	6.3	8.6	7.3	10.3	4.9	5.0	4.8	5.3
50	3.7	4.6	4.8	5.0	5.2	5.0	4.7	4.6	5.0	5.0	6.3	6.7	5.5	6.8	6.1	7.7	4.6	4.7	4.6	4.9
75	3.7	4.5	4.6	4.8	4.9	4.8	4.5	4.5	4.8	4.7	5.7	5.9	5.1	6.0	5.5	6.6	4.5	4.5	4.5	4.7
100	3.7	4.4	4.5	4.7	4.8	4.7	4.5	4.4	4.7	4.6	5.3	5.5	5.0	5.6	5.2	6.0	4.4	4.5	4.4	4.5
125	3.7	4.4	4.5	4.6	4.7	4.6	4.4	4.4	4.6	4.5	5.2	5.3	4.8	5.3	5.0	5.7	4.4	4.4	4.4	4.5
150	3.7	4.4	4.4	4.6	4.6	4.6	4.4	4.4	4.6	4.5	5.0	5.2	4.8	5.2	4.9	5.5	4.4	4.4	4.4	4.4
Tangent	3.6	4.2	4.2	4.4	4.4	4.4	4.2	4.2	4.4	4.2	4.4	4.4	4.4	4.4	4.4	4.4	4.2	4.2	4.2	4.2
Case II, One-Lane, One-Way Operation with Provision for Passing a Stalled Vehicle by Another of the Same Type																				
15	6.0	9.2	10.9	11.9	13.1	11.7	9.4	9.7	12.4	11.8	25.2	—	15.4	—	20.9	—	9.2	9.3	8.7	11.0
25	5.6	7.9	8.9	9.6	10.2	9.5	8.0	8.2	9.9	9.3	15.0	16.8	11.2	16.9	13.5	21.7	7.9	7.9	7.6	8.9
30	5.5	7.6	8.4	9.0	9.5	9.0	7.7	7.8	9.3	8.8	13.4	14.8	10.4	14.9	12.2	18.4	7.6	7.6	7.4	8.4
50	5.3	7.0	7.5	8.0	8.3	7.9	7.0	7.1	8.1	7.7	10.4	11.2	8.7	11.2	9.8	13.1	7.0	7.0	6.8	7.5
75	5.2	6.7	7.0	7.4	7.6	7.4	6.7	6.8	7.5	7.1	9.1	9.6	7.9	9.6	8.6	10.8	6.7	6.7	6.6	7.0
100	5.2	6.5	6.8	7.2	7.3	7.1	6.6	6.6	7.2	6.9	8.4	8.8	7.5	8.8	8.1	9.7	6.5	6.5	6.5	6.8
125	5.1	6.4	6.6	7.0	7.1	7.0	6.5	6.5	7.1	6.7	8.0	8.3	7.3	8.3	7.7	9.0	6.4	6.4	6.4	6.6
150	5.1	6.4	6.5	6.9	7.0	6.9	6.4	6.4	7.0	6.6	7.7	8.0	7.2	8.0	7.5	8.6	6.4	6.4	6.3	6.5
Tangent	5.0	6.1	6.1	6.4	6.4	6.4	6.1	6.1	6.4	6.1	6.4	6.4	6.4	6.4	6.4	6.4	6.1	6.1	6.1	6.1
Case III, Two-Lane Operation, Either One- or Two-Way (Same Type Vehicle in Both Lanes)																				
15	7.8	11.0	12.7	13.7	14.9	13.5	11.2	11.5	14.2	13.6	27.0	—	17.2	—	22.7	—	11.0	11.1	10.5	12.8
25	7.4	9.7	10.7	11.4	12.0	11.3	9.8	10.0	11.7	11.1	16.8	18.6	13.0	18.7	15.3	23.5	9.7	9.7	9.4	10.7
30	7.3	9.4	10.2	10.8	11.3	10.8	9.5	9.6	11.1	10.6	15.2	16.6	12.2	16.7	14.0	20.2	9.4	9.4	9.2	10.2
50	7.1	8.8	9.3	9.8	10.1	9.7	8.8	8.9	9.9	9.5	12.2	13.0	10.5	13.0	11.6	14.9	8.8	8.8	8.6	9.3
75	7.0	8.5	8.8	9.2	9.4	9.2	8.5	8.6	9.3	8.9	10.9	11.4	9.7	11.4	10.4	12.6	8.5	8.5	8.4	8.8
100	7.0	8.3	8.6	9.0	9.1	8.9	8.4	8.4	9.0	8.7	10.2	10.6	9.3	10.6	9.9	11.5	8.3	8.3	8.3	8.6
125	6.9	8.2	8.4	8.8	8.9	8.8	8.3	8.3	8.9	8.5	9.8	10.1	9.1	10.1	9.5	10.8	8.2	8.2	8.2	8.4
150	6.9	8.2	8.3	8.7	8.8	8.7	8.2	8.2	8.8	8.4	9.5	9.8	9.0	9.8	9.3	10.4	8.2	8.2	8.1	8.3
Tangent	6.8	7.9	7.9	8.2	8.2	8.2	7.9	7.9	8.2	7.9	8.2	8.2	8.2	8.2	8.2	8.2	7.9	7.9	7.9	7.9

Table 3-28b. Derived Pavement Widths for Turning Roadways for Different Design Vehicles

U.S. Customary																				
Radius on Inner Edge of Pavement, *R* (ft)	Case I, One-Lane Operation, No Provision for Passing a Stalled Vehicle																			
	P	SU-30	SU-40	BUS-40	BUS-45	CITY-BUS	S-BUS-36	S-BUS-40	A-BUS-11	WB-40	WB-62	WB-67	WB-67D	WB-92D	WB-100T	WB-109D	MH	P/T	P/B	MH/B
50	13	18	21	22	23	21	19	18	22	23	44	57	29	—	37	—	18	19	18	21
75	13	17	18	19	20	19	17	17	19	20	30	33	23	34	27	43	17	17	17	19
100	13	16	17	18	19	18	16	16	18	18	25	28	21	28	24	34	16	16	16	17
150	12	15	16	17	17	17	16	15	17	17	22	23	19	23	21	27	15	16	15	16
200	12	15	16	16	17	16	15	15	16	16	20	21	18	21	19	23	15	15	15	16
300	12	15	15	16	16	16	15	15	16	15	18	19	17	19	17	20	15	15	15	15
400	12	15	15	15	16	15	15	15	15	15	17	18	16	18	17	19	15	15	14	15
500	12	14	15	15	15	15	14	14	15	15	17	17	16	17	16	18	14	14	14	15
Tangent	12	14	14	15	15	15	14	14	15	14	15	15	15	15	15	15	14	14	14	14
Case II, One-Lane, One-Way Operation with Provision for Passing a Stalled Vehicle by Another of the Same Type																				
50	20	30	36	39	42	38	31	32	40	39	81	109	50	—	67	—	30	30	28	36
75	19	27	30	32	35	32	27	28	34	32	53	59	39	60	47	79	27	27	26	30
100	18	25	27	30	31	29	25	26	30	29	44	48	34	48	40	60	25	25	24	28
150	18	23	25	27	28	27	23	24	27	26	36	38	29	39	33	45	23	23	23	25
200	17	22	24	25	26	25	23	23	26	24	32	34	27	34	30	39	22	22	22	24
300	17	22	22	24	24	24	22	22	24	23	28	30	25	30	27	33	22	22	21	23
400	17	21	22	23	24	23	21	21	23	22	26	27	24	27	25	30	21	21	21	22
500	17	21	21	23	23	23	21	21	23	22	25	26	23	26	25	28	21	21	21	21
Tangent	17	20	20	21	21	21	20	20	21	20	21	21	21	21	21	21	20	20	20	20
Case III, Two-Lane Operation, Either One- or Two-Way (Same Type Vehicle in Both Lanes)																				
50	26	36	42	45	48	44	37	38	46	45	87	115	56	—	73	—	36	36	34	42
75	25	33	36	38	41	38	33	34	40	38	59	65	45	66	53	85	33	33	32	36
100	24	31	33	36	37	35	31	32	36	35	50	54	40	54	46	66	31	31	30	34
150	24	29	31	33	34	33	29	30	33	32	42	44	35	45	39	51	29	29	29	31
200	23	28	30	31	32	31	29	29	32	30	38	40	33	40	36	45	28	28	28	30
300	23	28	28	30	30	30	28	28	30	29	34	36	31	36	33	39	28	28	27	29
400	23	27	28	29	30	29	27	27	29	28	32	33	30	33	31	36	27	27	27	28
500	23	27	27	29	29	29	27	27	29	28	31	32	29	32	31	34	27	27	27	27
Tangent	23	26	26	27	27	27	26	26	27	26	27	27	27	27	27	27	26	26	26	26

Design Values

The total width, W, for separate turning roadways at intersections is derived by the summation of the proper width elements. The separate formulas for width and values for lateral clearance, C, and the allowance for difficulty of driving on curves, Z, for each of three cases are shown in Figure 3-21. Values for track width, U, are obtained from Figure 3-17 and values for front overhang, F_A, from Figure 3-18. Values of U and F_A are read from the figure for the turning radius, R_T, which is closely approximated by adding the track width and proper clearances to the radius of the inner edge of the turning roadway.

When determining the width for Case I, a lateral clearance, C, of 1.2 m [4 ft] is considered appropriate. The allowance for difficulty of driving curves, Z, is constant, equal to about 0.6 m [2 ft] for all radii of 150 m [500 ft] or less. In this case, the front overhang, F_A, need not be considered because no passing of another vehicle is involved.

For Case II, the width involves U and C for the stopped vehicle and the U and C for the passing vehicle. To this is added extra width for the front overhang, F_A, of one vehicle and the rear overhang, F_B, (if any) of the other vehicle. The width of rear overhang for a passenger car is considered to be 0.15 m [0.5 ft]. F_B for truck design vehicles is 0. A total clearance of one-half the value of C in the other two cases is assumed (i.e., 0.6 m [2 ft] for the stopped vehicle and 0.6 m [2 ft] for the passing vehicle). Because passing the stalled vehicle is accomplished at low speeds, the extra width allowance, Z, is omitted.

All the width elements apply for Case III. To the values of U and F_A obtained from Figures 3-17 and 3-18, respectively, the lateral clearance, C, of 1.2 m [4 ft]; F_B of 0.15 m [0.5 ft] for passenger cars; and Z of 0.6 m [2 ft] is added to determine the total width.

The derived widths for various radii for each design vehicle are given in Table 3-28. For general design use, the recommended widths given in Table 3-28 seldom apply directly, because the turning roadways usually accommodate more than one type of vehicle. Even parkways designed primarily for P vehicles are used by buses and maintenance trucks. At the other extreme, few if any public highways are designed to fully accommodate the WB-19 [WB-62] or longer design vehicles. Widths needed for some combination of separate design vehicles become the practical design guide for intersecting roadways. Such design widths are given in Table 3-29 for three logical conditions of mixed traffic that are defined below. However, where the larger design vehicles such as the WB-19 or WB-33D [WB-62 or WB-109D] will be using a turning roadway or ramp, the facility should accommodate their turning paths for at least the Case I condition. Therefore, Case I widths for the appropriate design vehicle and radius shown in Table 3-28 should be checked to determine whether they exceed widths shown in Table 3-29. If they do, consideration should be given to using the widths for Case I shown in Table 3-28 as the minimum widths for the turning roadway or ramp.

Traffic conditions for defining turning roadway widths are described in broad terms because data concerning the traffic volume, or the percentage of the total volume, for each type of vehicle are not available to define these traffic conditions precisely.

Traffic Condition A—This traffic condition consists predominantly of P vehicles, but some consideration is also given to SU-9 [SU-30] trucks; the values in Table 3-29 are somewhat higher than those for P vehicles in Table 3-28.

Traffic Condition B—This traffic condition includes sufficient SU-9 [SU-30] trucks to govern design, but some consideration is also given to tractor-semitrailer combination trucks; values in Table 3-29 for Cases I and III are those for SU vehicles in Table 3-28. For Case II, values are reduced as explained later in this section.

Traffic Condition C—This traffic condition includes sufficient tractor-semitrailer combination trucks, WB-12 [WB-40], to govern design; the values in Table 3-29 for Cases I and III are those for the WB-12 [WB-40] truck in Table 3-28. For Case II, values are reduced.

In general, Traffic Condition A may be assumed to have a small volume of trucks or only an occasional large truck; Traffic Condition B, a moderate volume of trucks (e.g., in the range of 5 to 10 percent of the total traffic); and Traffic Condition C, more and larger trucks.

Table 3-29. Design Widths of Pavements for Turning Roadways

Metric

Radius on Inner Edge of Pavement, *R* (m)	Pavement Width (m)								
	Case I One-Lane, One-Way Operation—no provision for passing stalled vehicle			Case II One-Lane, One-Way Operation—with provision for passing stalled vehicle			Case III Two-Lane Operation—either one-way or two-way operation		
	Design Traffic Conditions								
	A	B	C	A	B	C	A	B	C
15	5.4	5.5	7.0	6.0	7.8	9.2	9.4	11.0	13.6
25	4.8	5.0	5.8	5.6	6.9	7.9	8.6	9.7	11.1
30	4.5	4.9	5.5	5.5	6.7	7.6	8.4	9.4	10.6
50	4.2	4.6	5.0	5.3	6.3	7.0	7.9	8.8	9.5
75	3.9	4.5	4.8	5.2	6.1	6.7	7.7	8.5	8.9
100	3.9	4.5	4.8	5.2	5.9	6.5	7.6	8.3	8.7
125	3.9	4.5	4.8	5.1	5.9	6.4	7.6	8.2	8.5
150	3.6	4.5	4.5	5.1	5.8	6.4	7.5	8.2	8.4
Tangent	3.6	4.2	4.2	5.0	5.5	6.1	7.3	7.9	7.9

Width Modification for Edge Conditions			
No stabilized shoulder	None	None	None
Sloping curb	None	None	None
Vertical curb:			
one side	Add 0.3 m	None	Add 0.3 m
two sides	Add 0.6 m	Add 0.3 m	Add 0.6 m
Stabilized shoulder, one or both sides	Lane width for conditions B & C on tangent may be reduced to 3.6 m where shoulder is 1.2 m or wider	Deduct shoulder width(s); minimum pavement width as under Case I	Deduct 0.6 m where shoulder is 1.2 m or wider

Note:

A = predominantly P vehicles, but some consideration for SU trucks

B = sufficient SU-9 vehicles to govern design, but some consideration for semitrailer combination trucks

C = sufficient bus and combination-trucks to govern design

U.S. Customary

Radius on Inner Edge of Pavement, *R* (ft)	Pavement Width (ft)								
	Case I One-Lane, One-Way Operation—no provision for passing stalled vehicle			Case II One-Lane, One-Way Operation—with provision for passing stalled vehicle			Case III Two-Lane Operation—either one-way or two-way operation		
	Design Traffic Conditions								
	A	B	C	A	B	C	A	B	C
50	18	18	23	20	26	30	31	36	45
75	16	17	20	19	23	27	29	33	38
100	15	16	18	18	22	25	28	31	35
150	14	15	17	18	21	23	26	29	32
200	13	15	16	17	20	22	26	28	30
300	13	15	15	17	20	22	25	28	29
400	13	15	15	17	19	21	25	27	28
500	12	15	15	17	19	21	25	27	28
Tangent	12	14	14	17	18	20	24	26	26

Width Modification for Edge Conditions			
No stabilized shoulder	None	None	None
Sloping curb	None	None	None
Vertical curb:			
one side	Add 1 ft	None	Add 1 ft
two sides	Add 2 ft	Add 1 ft	Add 2 ft
Stabilized shoulder, one or both sides	Lane width for conditions B & C on tangent may be reduced to 12 ft where shoulder is 4 ft or wider	Deduct shoulder width(s); minimum pavement width as under Case I	Deduct 2 ft where shoulder is 4 ft or wider

Note:

A = predominantly P vehicles, but some consideration for SU trucks

B = sufficient SU-30 vehicles to govern design, but some consideration for semitrailer combination trucks

C = sufficient bus and combination-trucks to govern design

In Table 3-29, smaller vehicles in combination are assumed for deriving Case II widths than for deriving Case III widths because passing of stalled vehicles in the former is apt to be very infrequent. Moreover, full offtracking need not be assumed for both the stalled and the passing vehicles. Often the stalled vehicles will be adjacent to the inner edge of roadway, thereby providing additional clearance for the passing vehicle.

The design vehicles or combinations of different design vehicles used in determination of values given in Table 3-29 for the three traffic conditions, assuming full clearance for the design vehicles indicated, are as follows:

	Metric			U.S. Customary		
	Design Traffic Condition			Design Traffic Condition		
Case	A	B	C	A	B	C
I	P	SU-9	WB-12	P	SU-30	WB-40
II	P–P	P–SU-9	SU-9–SU-9	P–P	P–SU-30	SU-30–SU-30
III	P–SU-9	SU-9–SU-9	WB-12–WB-12	P–SU-30	SU-30–SU-30	WB-40–WB-40

The combination of letters, such as P–SU-9 [SU-30] for Case II, means that the design width in this example allows a P design vehicle to pass a stalled SU-9 [SU-30] design truck or vice versa. In assuming full clearance, allowance was made for the values of C as discussed.

In negotiating roadways designed for smaller vehicles, larger vehicles will have less clearance, will need to use lower speeds, and will demand more caution and skill by drivers, but there is a limit to the size of vehicles that can be operated on these narrower roadways. The larger vehicles that can be operated on turning roadways of the widths shown in Table 3-29, but with partial clearance varying from about one-half the total values of *C*, as discussed for the sharper curves, to nearly full values for the flatter curves, are as follows:

	Metric			U.S. Customary		
	Design Traffic Condition			Design Traffic Condition		
Case	A	B	C	A	B	C
I	WB-12	WB-12	WB-19	WB-40	WB-40	WB-62
II	P–SU-9	P–WB-12	SU-9–WB-12	P–SU-30	P–WB-40	SU-30–WB-40
III	SU-9–WB-12	WB-12–WB-12	WB-19–WB-19	SU-30–WB-40	WB-40–WB-40	WB-62–WB-62

The widths in Table 3-29 are subject to some modification with respect to the treatment at the edge, as shown at the bottom of the table. An occasional large vehicle can pass another on a roadway designed for small vehicles if there is space and stability outside the roadway and there is no barrier to prevent its occasional use. In such cases, the width can be a little narrower than the tabulated dimension. Vertical curbs along the edge of a lane give drivers a sense of restriction, and occasional large vehicles have no additional space in which to maneuver; for this reason, such roadways should be a little wider than the values shown in Table 3-29.

When there is an adjacent stabilized shoulder, the widths for Cases II and III and, under certain conditions, for Case I on roadways on tangent may be reduced. Case II values may be reduced by the additional width of stabilized shoulder but not below the widths for Case I. Similarly, Case III values may be reduced by 0.6 m [2 ft]. Case I values for the individual design vehicles are recommended minimums and further reduction is not in order, even with a usable shoulder, except on tangents. When vertical curbs are used on both sides, the tabulated widths should be increased by 0.6 m [2 ft] for Cases I and III, or by 0.3 m [1 ft] for Case II, because stalled vehicles are passed at low speed. Where such a curb is on only one side of the roadway, the added width may be only 0.3 m [1 ft] for Cases I and III, and no added width is needed for Case II.

The use of Table 3-29 in design is illustrated by the following example. Assume that the geometric layout and traffic volume for a specific turning movement are such that one-lane, one-way operation with need for passing a stalled vehicle is appropriate (Case II), and that the traffic volume includes 10 to 12 percent trucks with an occasional large semitrailer combination for which Traffic Condition C is deemed applicable. Then, with a radius of 50 m [165 ft] for the inner edge of the traveled way, the width tabulated in Table 3-29 is 7.0 m [23 ft]. With a 1.2-m [4-ft] stabilized shoulder, the turning roadway width may be reduced to 5.8 m [19 ft] (see lower part of Table 3-29). With a vertical curb on each side (and therefore, with no stabilized shoulder present), the turning roadway width should be not less than 7.3 m [24 ft].

Widths Outside the Traveled Way

The roadway width for a turning roadway includes the shoulders or equivalent lateral clearance outside the traveled way. Over the whole range of intersections, the appropriate shoulder width varies from none, or minimal, on curbed urban streets to the width of an open-highway cross section. The more general cases are discussed in this section.

Within a channelized intersection, shoulders for turning roadways are usually unnecessary. The lanes may be defined by curbs, pavement markings, or islands. The islands may be curbed and the general dimensional controls for islands provide the appropriate lateral clearances outside the edges of the turning roadway. In most instances, the turning roadways are relatively short, and shoulder sections are not needed for the temporary storage of vehicles. A discussion of island dimensions can be found in Section 9.6.3.

Where there is a separate roadway for right turns, its left edge defines one side of the triangular island. If the island is small or especially important in directing movements, it may be defined both by curbs and pavement markings. On the other hand, where the turning radius is large, the side of the island may be defined by guideposts, by delineators, or simply by pavement markings and the edge of the pavement of the turning roadway. In any case, a developed left shoulder is normally unnecessary. However, there should be either an offset, if curbs are used, or a fairly level section of sufficient width on the left to avoid affecting the lateral placement of vehicles.

A shoulder usually is provided on the right side of a right-turning roadway in rural areas. In cross section and general treatment, the right shoulder should be essentially the same as the shoulder of the adjacent open-highway section, possibly somewhat reduced in width because of conditions at the intersections. Because turning vehicles have a tendency to encroach on the shoulder, consideration should be given to providing heavy-duty right shoulders to accommodate the associated wheel loads. Although a curb on the right side might reduce maintenance operations that result from vehicles hugging the inside of the curve and causing edge depressions or raveling, the introduction of curbing adjacent to high-speed highways should be discouraged. For low-speed urban conditions, curbing of the right edge of a turning roadway is normal practice. Curbs are discussed in greater detail in Chapter 4.

On large-scale channelized layouts and at interchanges, there may be turning roadways of sufficient curvature and length to be well removed from other roadways. Such turning roadways should have a shoulder on both sides. Curbs, when used, should be located at the outside edge of the shoulder and should be sloping.

Some turning roadways, particularly ramps, pass over drainage structures, pass over or under other roadways, or pass adjacent to walls or rock cuts on one or both sides. For such locations, the minimum clearances for structures, as established in later chapters and in the current edition of the *AASHTO LRFD Bridge Design Specifications* (*7*), apply directly. In addition, the design should be evaluated for adequate sight distance, because the sharp curve may need above-minimum lateral clearance.

Table 3-30 is a summary of the range of design values for the general turning roadway conditions previouslydescribed. On roadways without curbs or with sloping curbs, the adjacent shoulder should be of the same type and cross section as that on the approach highway. The widths shown are for usable shoulders. Where roadside barriers are provided, the width indicated should be measured to the face of the barrier, and the graded width should be about 0.6 m [2.0 ft] greater. For other than low-volume conditions, it is desirable that right shoulders be surfaced or otherwise stabilized for a width of 1.2 m [4.0 ft] or more.

Table 3-30. Range of Usable Shoulder Widths or Equivalent Lateral Clearances Outside of Turning Roadways, Not on Structure

	Metric		U.S. Customary	
Turning Roadway Condition	Shoulder Width or Lateral Clearance Outside of Traveled-Way Edge (m)		Shoulder Width or Lateral Clearance Outside of Traveled-Way Edge (ft)	
	Left	Right	Left	Right
Short length, usually within channelized intersection	0.6 to 1.2	0.6 to 1.2	2 to 4	2 to 4
Intermediate to long length or in cut or on fill	1.2 to 3.0	1.8 to 3.6	4 to 10	6 to 12

Note: All dimensions should be increased, where appropriate, for sight distance.

3.3.12 Sight Distance on Horizontal Curves

Another element of horizontal alignment is the sight distance across the inside of curves. Where there are sight obstructions (such as walls, cut slopes, buildings, and longitudinal barriers) on the inside of curves or the inside of the median lane on divided highways and their removal to increase sight distance is impractical, a design may need adjustment in the normal highway cross section or the alignment. Because of the many variables in alignment, in cross section, and in the number, type, and location of potential obstructions, specific study is usually needed for each individual curve. With sight distance for the design speed as a control, the designer should check the actual conditions on each curve and make the appropriate adjustments to provide adequate sight distance.

Stopping Sight Distance

For general use in design of a horizontal curve, the sight line is a chord of the curve, and the stopping sight distance is measured along the centerline of the inside lane around the curve. Figure 3-22 is a design chart showing the horizontal sight line offsets needed for clear sight areas that satisfy stopping sight distance criteria presented in Table 3-1 for horizontal curves of various radii on flat grades. Figure 3-22 includes radii for all superelevation rates to a maximum of 12 percent.

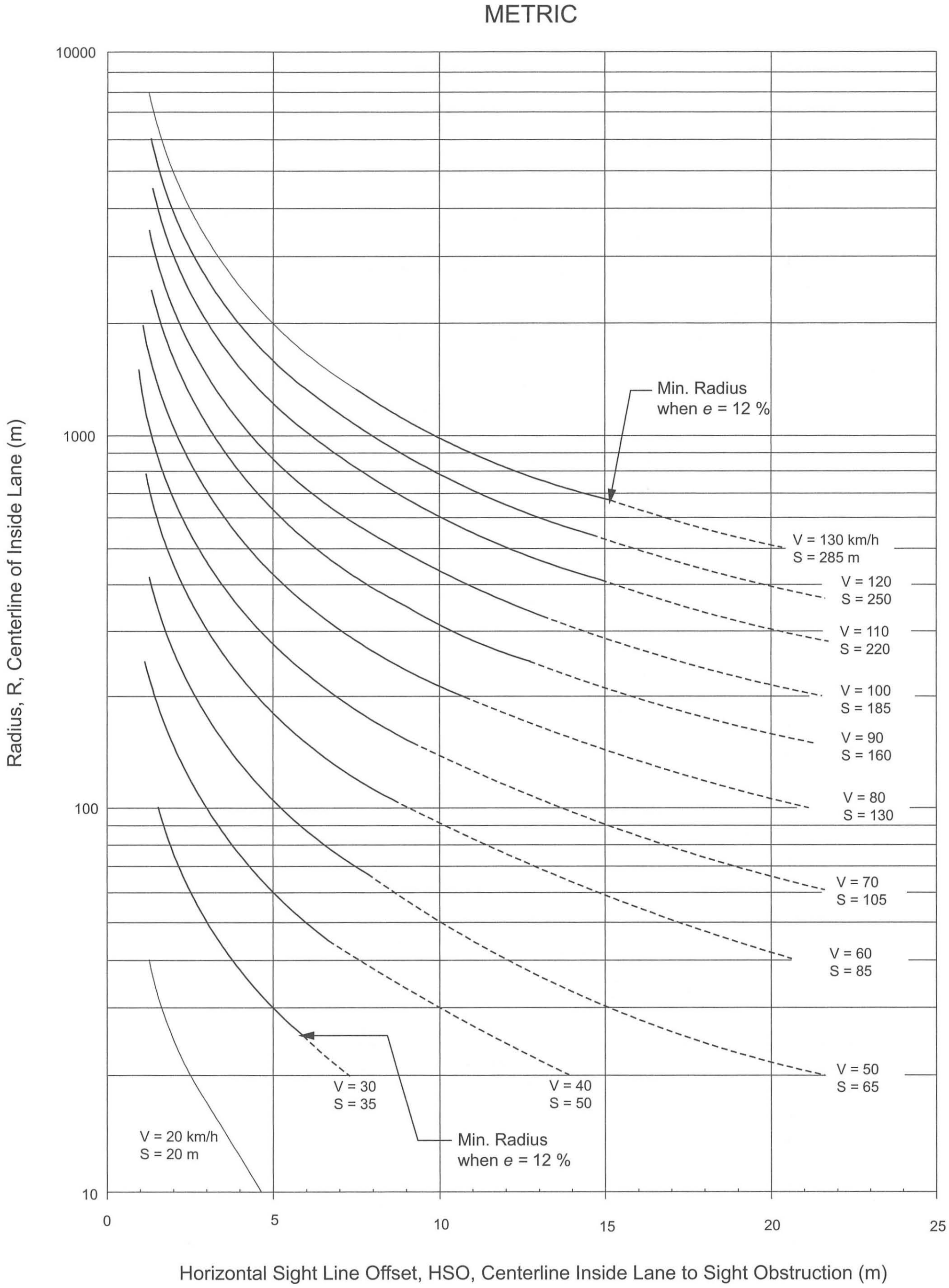


Figure 3-22a. Design Controls for Stopping Sight Distance on Horizontal Curves

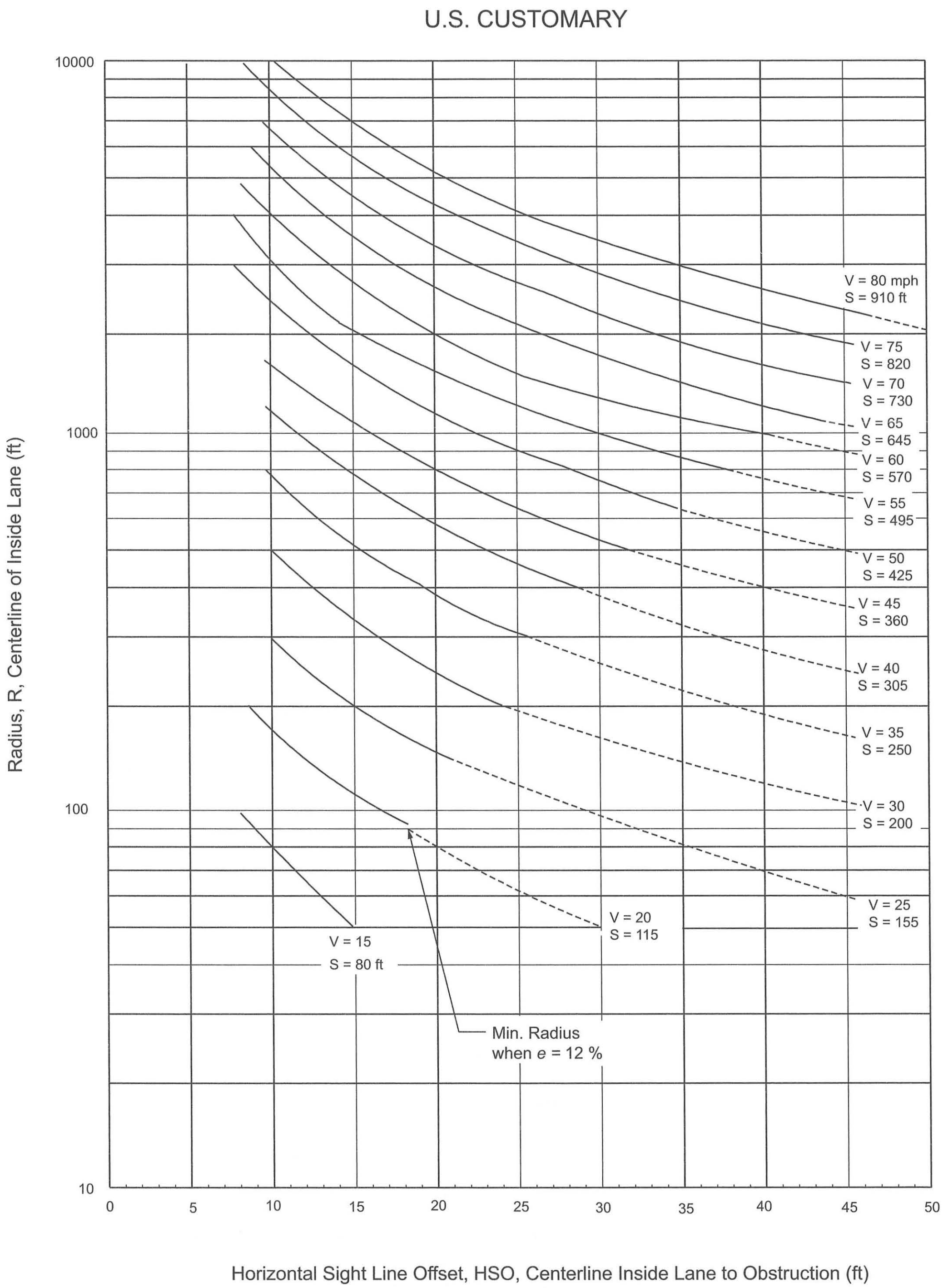


Figure 3-22b. Design Controls for Stopping Sight Distance on Horizontal Curves

The horizontal sight line offset (*HSO*) values in Figure 3-22 are derived from geometry for the several dimensions, as indicated in the diagrammatic sketch in Figure 3-23 and in Equation 3-36. The equation applies only to circular curves longer than the sight distance for the pertinent design speed. The relationships between R, HSO, and V in this chart can be quickly checked. For example, with an 80-km/h [50-mph] design speed and a curve with a 350-m [1,150-ft] radius, a clear sight area with a horizontal sight line offset of approximately 6.0 m [20 ft] is needed for stopping sight distance. As another example, for a sight obstruction at a distance HSO equal to 6.0 m [20 ft] from the centerline of the inside lane on a curve with a 175-m [575-ft] radius, the sight distance needed is approximately at the upper end of the range for a speed of approximately 60 km/h [40 mph].

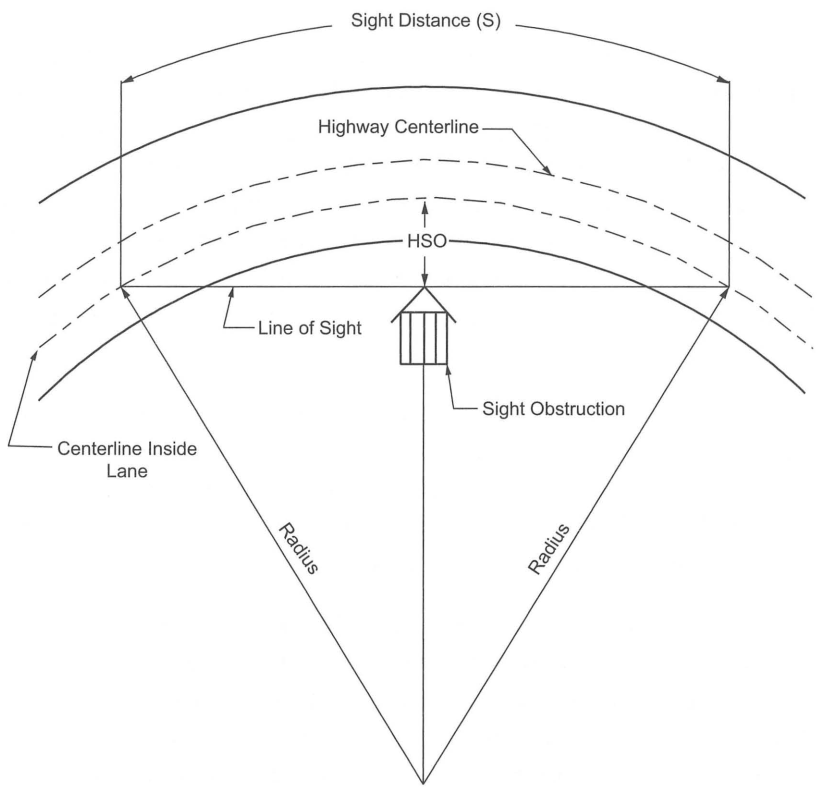


Figure 3-23. Diagram Illustrating Components for Determining Horizontal Sight Distance

Metric	U.S. Customary
$HSO = R\left[1 - \cos\left(\frac{28.65S}{R}\right)\right]$	$HSO = R\left[1 - \cos\left(\frac{28.65S}{R}\right)\right]$ (3-36)
where:	where:
HSO = Horizontal sight line offset, m	HSO = Horizontal sight line offset, ft
S = Stopping sight distance, m	S = Stopping sight distance, ft
R = Radius of curve, m	R = Radius of curve, ft

Horizontal sight restrictions may occur where there is a cut slope on the inside of the curve. For the 1.08-m [3.50-ft] eye height and the 0.60-m [2.00-ft] object height used for stopping sight distance, a height of 0.84 m [2.75 ft] may be used as the midpoint of the sight line where the cut slope usually obstructs sight. This assumes that there is little or no vertical curvature. For a highway with a 6.6-m [22-ft] traveled way, 1.2-m [4-ft] shoulders, an allowance of 1.2 m [4 ft] for a ditch section, and 1V:2H (1 m or 1 ft vertically for each, 2 m or 2 ft horizontally) cut slopes, the sight obstruction is approximately 5.75 m [19 ft] outside the centerline of the inside lane. This is sufficient for adequate sight distance at 50 km/h [30 mph] when curves have a radius of about 90 m [275 ft] or more and at 80 km/h [50 mph] when curves have a radius of about 375 m [1,230 ft] or more. Curves sharper than these would need flatter slopes, benching, or other adjustments. At the other extreme, highways with normal lateral dimensions of more than 16 m [52 ft] provide adequate stopping sight distances for horizontal curves over the entire range of design speeds and curves.

In some instances, retaining walls, concrete median barriers, and other similar features constructed on the inside of curves may be sight obstructions and should be checked for stopping sight distance. As an example, an obstruction of this type, located 1.2 m [4 ft] from the inside edge of a 7.2-m [24-ft] traveled way, has a horizontal sight line offset of approximately 3.0 m [10 ft]. At 80 km/h [50 mph], this provides sufficient sight distance when a curve has a radius of about 700 m [2,300 ft] or more. If the obstruction is moved an additional 0.3 m [1 ft] away from the roadway creating a horizontal sight line offset of 3.3 m [11 ft], a curve with a radius of 625 m [2,000 ft] or more provides sufficient sight distance at the same 80 km/h [50 mph] speed. The same finding would be applicable to existing buildings or similar sight obstructions on the inside of curves.

Where sufficient stopping sight distance is not available because a railing or a longitudinal barrier constitutes a sight obstruction, alternative designs should be considered. The alternatives are: (1) increase the offset to the obstruction, (2) increase the radius, or (3) reduce the design speed. However, the alternative selected should not incorporate shoulder widths on the inside of the curve in excess of 3.6 m [12 ft] because of the concern that drivers will use wider shoulders as a passing or travel lane.

As can be seen from Figure 3-23, the method presented is only exact when both the vehicle and the sight obstruction are located within the limits of the simple horizontal curve. When either the vehicle or the sight obstruction is situated beyond the limits of the simple curve, the values obtained are only approximate. The same is true if either the vehicle, the sight obstruction, or both are situated within the limits of a spiral or a compound curve. In these instances, the value obtained would result in horizontal sight line offset values slightly larger than those needed to satisfy the desired stopping sight distance. In many instances, the resulting additional clearance will not be significant. Whenever Figure 3-22 is not applicable, the design should be checked either by utilizing graphical procedures or by utilizing a computational method. Reference (*50*) provides a computational method for making such checks.

Passing Sight Distance

The minimum passing sight distance for a two-lane road or street is about twice the minimum stopping sight distance at the same design speed. To conform to those greater sight distances, clear sight areas on the inside of curves should have widths in excess of those discussed. Equation 3-36 is directly applicable to passing sight distance but is of limited practical value except on long curves. A chart demonstrating use of this equation would primarily add value for reaching negative conclusions—that it would be difficult to maintain passing sight distance on other than very flat curves.

Passing sight distance is measured between an eye height of 1.08 m [3.50 ft] and an object height of 1.08 m [3.50 ft]. The sight line near the center of the area inside a curve is approximately 0.24 m [0.75 ft] higher than for stopping sight distance. In cut sections, the resultant lateral dimension for normal highway cross sections (1V:2H to 1V:6H backslopes) between the centerline of the inside lane and the midpoint of the sight line is from 0.5 to 1.5 m [1.5 to 4.5 ft] greater than that for stopping sight distance. It is obvious that for many cut sections, design for passing sight distance should, for practical reasons, be limited to tangents and very flat curves. Even in level terrain, provision of passing sight distance would need a clear area inside each curve that would, in some instances, extend beyond the normal right-of-way line.

In general, the designer should use graphical methods to check sight distance on horizontal curves. This method is presented in Figure 3-2 and described in the accompanying discussion.

3.3.13 General Controls for Horizontal Alignment

In addition to the specific design elements for horizontal alignment discussed under previous headings, a number of general controls are recognized in practice. These controls are not subject to theoretical derivation, but they are important for efficient and smooth-flowing highways. Excessive curvature or poor combinations of curvature limit traffic capacity, cause economic losses from increased travel time and operating costs, and detract from a pleasing appearance. To avoid these poor design practices, the general controls that follow should be used where practical:

- Alignment should be as directional as practical, but should be consistent with the topography and help preserve developed properties and community values. A flowing line that conforms generally to the natural contours is preferable to one with long tangents that slashes through the terrain. With curvilinear alignment, construction scars can be kept to a minimum and natural slopes and growth can be preserved. Such design is desirable from a construction and maintenance standpoint. In general, the number of short curves should be kept to a minimum. Winding alignment composed of short curves should be avoided because it usually leads to erratic operation. Although the aesthetic qualities of curving alignment are important, long tangents are needed on two lane highways so that sufficient passing sight distance is available on as much of the highway length as practical.

- In alignment developed for a given design speed, the minimum radius of curvature for that speed should be avoided wherever practical. The designer should attempt to use generally flat curves, saving the minimum radius for the most critical conditions. In general, the central angle of each curve should be as small as the physical conditions permit, so that the highway will be as directional as practical. This central angle should be absorbed in the longest practical curve, but on two-lane highways, the exception noted in the preceding paragraph applies to preserve passing sight distance.

- Consistent alignment should always be sought. Sharp curves should not be introduced at the ends of long tangents. Sudden changes from areas of flat curvature to areas of sharp curvature should be avoided. Where sharp curvature is introduced, it should be approached, where practical, by a series of successively sharper curves.

- For small deflection angles, curves should be sufficiently long to avoid the appearance of a kink. Curves should be at least 150 m [500 ft] long for a central angle of 5 degrees, and the minimum length should be increased 30 m [100 ft] for each 1-degree decrease in the central angle. The minimum length for horizontal curves on main highways, $L_{c\,\mathrm{min}}$, should be three times the design speed expressed in km/h [15 times the design speed expressed in mph], or $L_{c\,\mathrm{min}} = 3V$ [$15V$]. On high-speed controlled-access facilities that use flat curvature for aesthetic reasons, the desirable minimum length for curves should be double the minimum length described above, or $L_{c\,\mathrm{des}} = 6V$ [$30V$].

- Sharp curvature should be avoided on long, high fills. In the absence of cut slopes, shrubs, and trees that extend above the level of the roadway, it is difficult for drivers to perceive the extent of curvature and adjust their operation accordingly.

- Caution should be exercised in the use of compound circular curves. While the use of compound curves affords flexibility in fitting the highway to the terrain and other ground controls, the ease with which such curves can be used may tempt the designer to use them without restraint. Preferably their use should be avoided where curves are sharp. Compound curves with large differences in radius introduce the same concerns that arise at tangent approaches to circular curves. Where topography or right-of-way restrictions make their use appropriate, the radius of the flatter circular arc, R_1, should not be more than 50 percent greater than the radius of the sharper circular arc, R_2 (i.e., R_1 should not exceed 1.5 R_2). A multiple compound curve (i.e., several curves in sequence) may be suitable as a transition to sharp curves as discussed in "Compound Circular Curves" of Section 3.3.8. A spiral transition between flat curves and sharp curves may be desirable. On one-way roads, such as ramps, the difference in radii of compound curves is not so important if the second curve is flatter than the first. However, the use of compound curves on ramps, with a flat curve between two sharper curves, is not good practice.

- Abrupt reversals in alignment should be avoided. Such changes in alignment make it difficult for drivers to keep within their own lane. It is also difficult to superelevate both curves adequately, and erratic operation may result. The distance between reverse curves should be the sum of the superelevation runoff lengths and the tangent runout lengths or, preferably, an equivalent length with spiral curves, as defined in Section 3.3.8, "Transition Design Controls." If sufficient distance (i.e., more than 100 m [300 ft]) is not available to permit the tangent runout lengths or preferably an equivalent length with spiral to return to a normal crown section, there may be a long length where the centerline and the edges of roadway are at the same elevation and poor transverse drainage is likely. In this case, the superelevation runoff lengths should be increased until they adjoin, thus providing one instantaneous level section. For traveled ways with straight cross slopes, there is less difficulty in returning the edges of roadway to a normal section and the 100-m [300-ft] guideline discussed above may be decreased.

- The "broken-back" or "flat-back" arrangement of curves (with a short tangent between two curves in the same direction) should be avoided except where very unusual topographical or right-of-way conditions make other alternatives impractical. Except on circumferential highways, most drivers do not expect successive curves to be in the same direction; the preponderance of successive curves in opposite directions may develop a subconscious expectation among drivers that makes successive curves in the same direction unexpected. Broken-back alignments are also not pleasing in appearance. Use of spiral transitions or compound curve alignments, in which there is some degree of continuous superelevation, is preferable for such situations. The term "broken-back" usually is not applied when the connecting tangent is of considerable length. Even in this case, the alignment may be unpleasant in appearance when both curves are clearly visible for some distance ahead.

- To avoid the appearance of inconsistent distortion, the horizontal alignment should be coordinated carefully with the profile design. General controls for this coordination are discussed in Section 3.5 on "Combinations of Horizontal and Vertical Alignment."

- Changing median widths on tangent alignments should be avoided, where practical, so as not to introduce a distorted appearance.

3.4 VERTICAL ALIGNMENT

3.4.1 Terrain

The topography of the land traversed has an influence on the alignment of roads and streets. Topography affects horizontal alignment, but has an even more pronounced effect on vertical alignment. To characterize variations in topography, engineers generally separate it into three classifications according to terrain—level, rolling, and mountainous.

In level terrain, highway sight distances, as governed by both horizontal and vertical restrictions, are generally long or can be made to be so without construction difficulty or major expense.

In rolling terrain, natural slopes consistently rise above and fall below the road or street grade, and occasional steep slopes offer some restriction to normal horizontal and vertical roadway alignment.

In mountainous terrain, longitudinal and transverse changes in the elevation of the ground with respect to the road or street are abrupt, and benching and side hill excavation are frequently needed to obtain acceptable horizontal and vertical alignment.

Terrain classifications pertain to the general character of a specific route corridor. Routes in valleys, passes, or mountainous areas that have all the characteristics of roads or streets traversing level or rolling terrain should be classified as level or rolling. In general, rolling terrain generates steeper grades than level terrain, causing trucks to reduce speeds below those of passenger cars; mountainous terrain has even greater effects, causing some trucks to operate at crawl speeds.

3.4.2 Grades

Highways and streets should be designed to encourage uniform operation throughout. As discussed in Sections 2.3.6, 3.2, and 3.3, design speeds are used as a means toward this end by correlation of various geometric features of the road or street. Design criteria have been determined for many highway features, but little is known about the appropriate relationship of roadway grades to design speed. Vehicle operating characteristics on grades and established relationships of grades and their lengths to design speed are presented in this section.

Vehicle Operating Characteristics on Grades

Passenger cars—The practices of passenger car drivers on grades vary greatly, but it is generally accepted that nearly all passenger cars can readily negotiate grades as steep as 4 to 5 percent without an appreciable loss in speed below that normally maintained on level roadways. Speed loss may be more pronounced for cars with high weight/power ratios, including some compact and subcompact cars.

Studies show that, under uncongested conditions, operation on a 3 percent upgrade has only a slight effect on passenger car speeds compared to operations on level terrain. On steeper upgrades, speeds decrease progressively with increases in the grade. On downgrades, passenger car speeds generally are slightly higher than on level sections, but local conditions govern.

Trucks—The effect of grades on truck speeds is much more pronounced than on speeds of passenger cars. The average speed of trucks on level sections of highway approximates the average speed of 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 (*18*, *24*, *36*, *37*, *52*, *61*, *67*).

The effect of rate and length of grade on the speed of a typical heavy truck is shown in Figures 3-24 and 3-25. From Figure 3-24 it can be determined how far a truck, starting its climb from any speed up to approximately 120 km/h [70 mph], travels up various grades or combinations of grades before a certain or uniform speed is reached. For instance, with an entering speed of approximately 110 km/h [70 mph], the truck travels about 950 m [2,700 ft] up a 6 percent grade before its speed is reduced to 60 km/h [35 mph]. If the entering speed is 60 km/h [35 mph], the speed at the end of a 300-m [1,000-ft] climb is about 43 km/h [26 mph]. This is determined by starting on the curve for a 6 percent grade corresponding to 60 km/h [35 mph] for which the distance is 750 m [2,500 ft], and proceeding along it to the point where the distance is 300 m [1,000 ft] more, or 1 050 m [3,500 ft], for which the speed is about 43 km/h [26 mph]. Figure 3-24 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 40 km/h [25 mph] or more only on grades of less than 3.5 percent. 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.

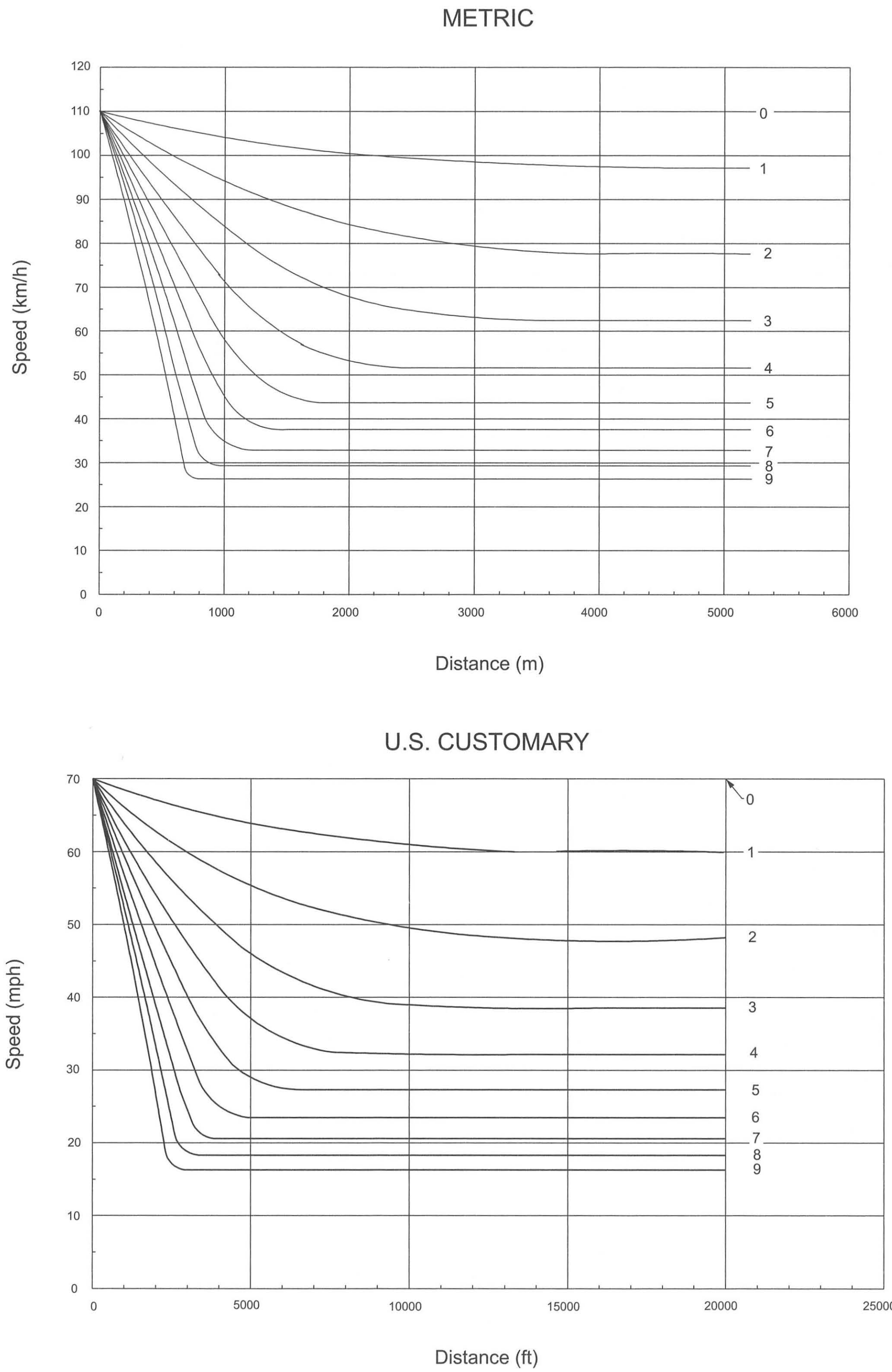


Figure 3-24. Speed-Distance Curves for a Typical Heavy Truck of 120 kg/kW [200 lb/hp] for Deceleration on Upgrades

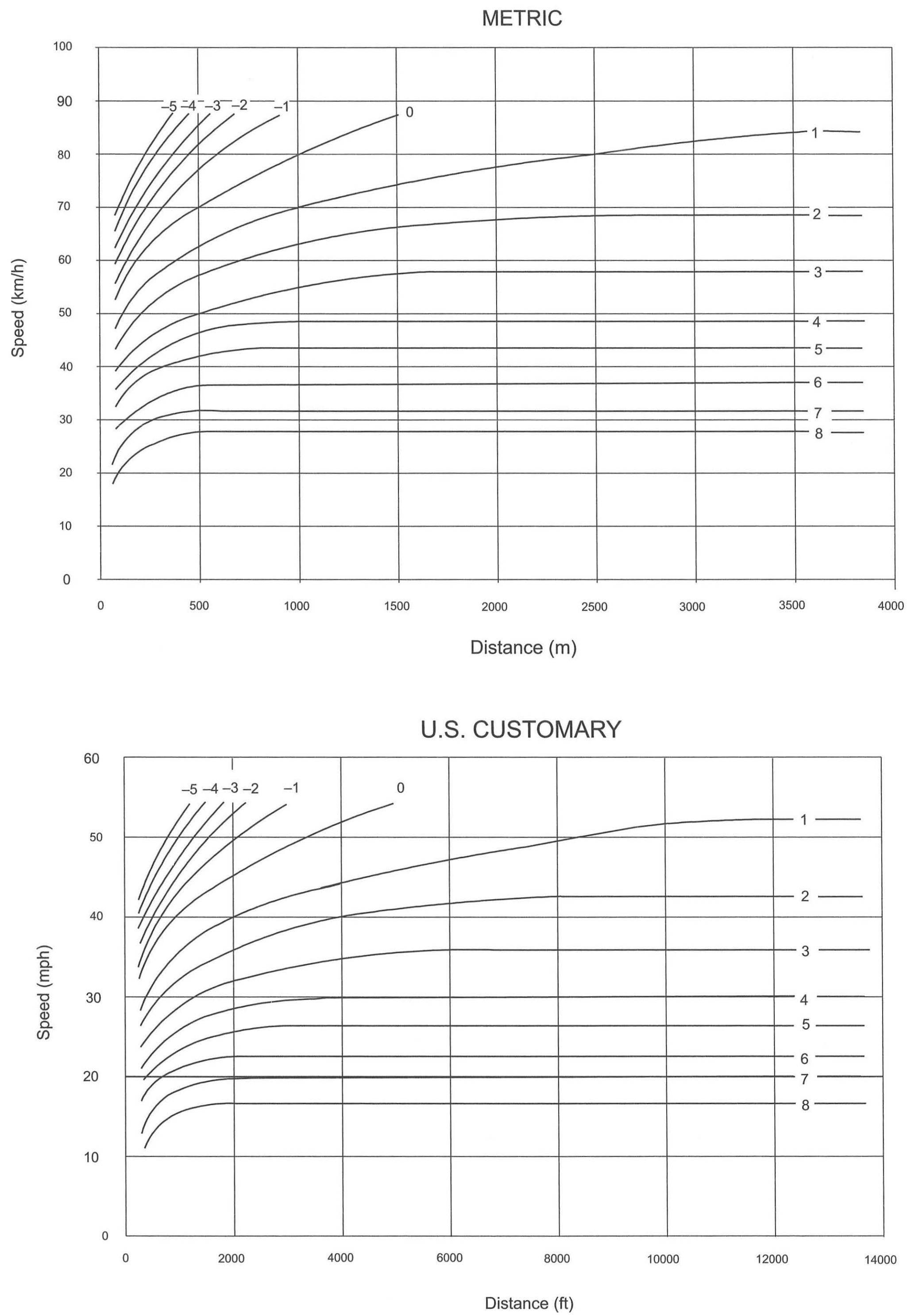


Figure 3-25. Speed-Distance Curves for Acceleration of a Typical Heavy Truck of 120 kg/kW [200 lb/hp] on Upgrades and Downgrades

Travel time (and, therefore, speed) of trucks on grades is directly related to the weight/power ratio. Trucks of the same weight/power ratio typically have similar operating characteristics. Hence, this ratio is of considerable assistance in anticipating the performance of trucks. Normally, the weight/power ratio is expressed in terms of gross weight and net power, in units of kg/kW [wt/hp]; while the metric unit kg is a unit of mass, rather than weight, it is commonly used to represent the weight of object. It has been found that trucks with weight/power ratios of about 120 kg/kW [200 lb/hp] have acceptable operating characteristics from the standpoint of the highway user. Such a weight/power ratio should provide a minimum speed of about 60 km/h [35 mph] on a 3 percent upgrade. There is evidence that the automotive industry finds a weight/power ratio of this magnitude acceptable as a minimum goal in the design of commercial vehicles. There is also evidence that carrier operators are voluntarily recognizing this ratio as the minimum performance control in the loads placed on trucks of different power, the overall result being that weight/power ratio of trucks on highways has improved in recent years. Ratios developed from information obtained in conjunction with the nationwide brake performance studies conducted between 1949 and 1985 show, for example, that for a gross vehicle weight of 18 000 kg [40,000 lb], the average weight/power ratio decreased from about 220 kg/kW [360 lb/hp] in 1949, to about 130 kg/kW [210 lb/hp] in 1975; the weight/power ratio continued to fall to about 80 kg/kW [130 lb/hp] in 1985. This decreased weight/power ratio means greater power and better climbing ability for trucks on upgrades.

There is a trend toward larger and heavier trucks with as many as three trailer units allowed on certain highways in some states. Studies indicate that as the number of axles increases, the weight/power ratio increases. Taking all factors into account, it appears conservative to use a weight/power ratio of 120 kg/kW [200 lb/hp] in determining critical length of grade. However, there are locations where a weight/power ratio as high as 120 kg/kW [200 lb/hp] is not appropriate. Where this occurs, designers are encouraged to utilize either a more representative weight/power ratio or an alternate method that more closely fits the conditions.

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 3-26 in the same manner as for trucks described in the immediately preceding paragraphs. 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 (*65*). The critical vehicle was considered to be a vehicle pulling a travel trailer, and the charts in Figure 3-26 for a typical recreational vehicle are based on that assumption.

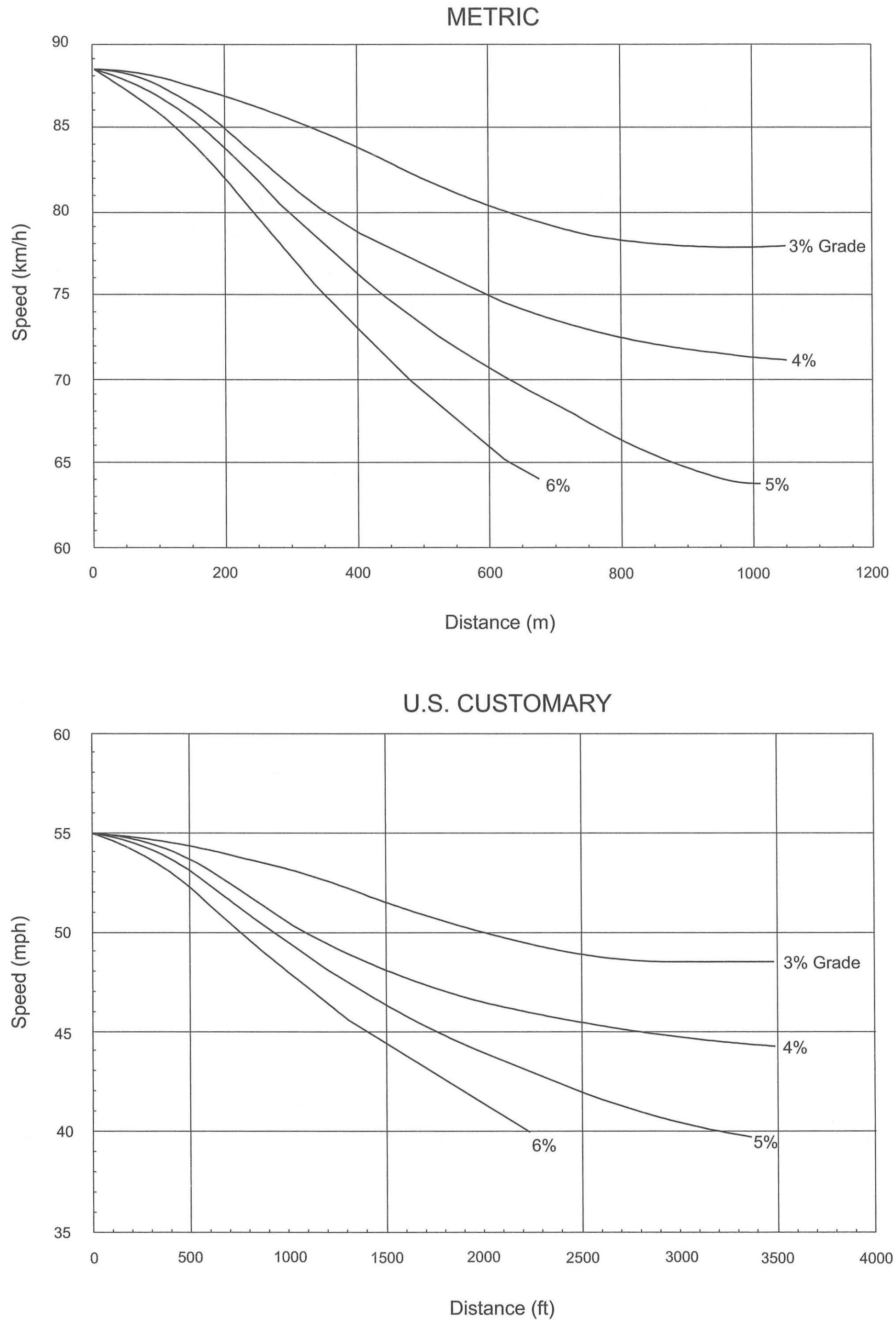


Figure 3-26. Speed-Distance Curves for a Typical Recreational Vehicle on the Selected Upgrades (*65*)

Control Grades for Design

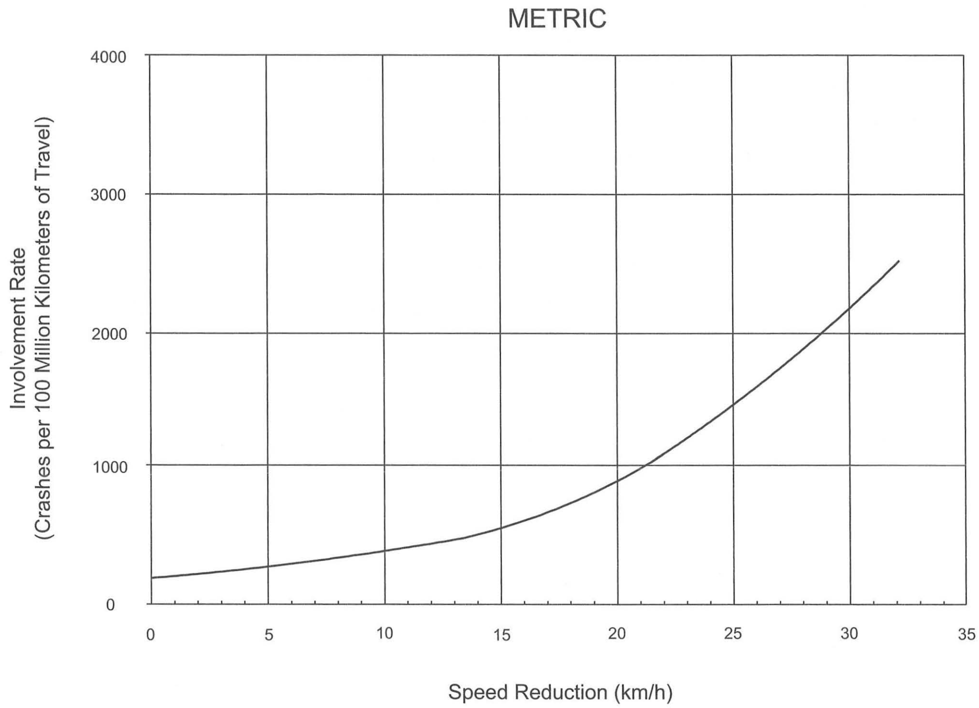
Maximum grades—On the basis of the data in Figures 3-24 through 3-27, and according to the grade controls now in use in a large number of states, reasonable design guidelines for maximum grades can be established. Maximum grades of about 5 percent are considered appropriate for a design speed of 110 km/h [70 mph]. For a design speed of 50 km/h [30 mph], maximum grades generally are in the range of 7 to 12 percent, depending on terrain. If only the more important highways are considered, it appears that maximum grades of 7 or 8 percent are representative of current design practice for a 50-km/h [30-mph] design speed. Control grades for design speeds from 60 to 100 km/h [40 to 60 mph] fall between the above extremes. Maximum grade controls for each functional class of highway and street are presented in Chapters 5 through 8.

The maximum design grade should be used only infrequently; in most cases, grades should be less than the maximum design grade. At the other extreme, for short grades less than 150 m [500 ft] in length and for one-way downgrades, the maximum grade may be about 1 percent steeper than other locations; for low-volume rural highways, the maximum grade may be 2 percent steeper.

Minimum grades—Flat grades can typically provide proper surface drainage on uncurbed highways where the cross slope is adequate to drain the pavement surface laterally. With curbed highways or streets, longitudinal grades should be provided to facilitate surface drainage. An appropriate minimum grade is typically 0.5 percent, but grades of 0.30 percent may be used where there is a paved surface accurately sloped and supported on firm subgrade. Use of even flatter grades may be justified in special cases as discussed in Chapter 5. Particular attention should be given to the design of stormwater inlets and their spacing to keep the spread of water on the traveled way within tolerable limits. Roadside channels and median swales frequently need grades steeper than the roadway profile for adequate drainage. Drainage channels are discussed in Section 4.8.3.

Critical Lengths of Grade for Design

Maximum grade in itself is not a complete design control. It is also appropriate to consider the length of a particular grade in relation to desirable vehicle operation. The term "critical length of grade" is used to indicate the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed. For a given length of grade, lengths less than critical result in acceptable operation in the desired range of speeds. If the desired freedom of operation is to be maintained on grades longer than critical, design adjustments such as changes in location to reduce grades or addition of extra lanes should be considered. The data for critical lengths of grade should be used with other pertinent factors (such as traffic volume in relation to capacity) to determine where added lanes are warranted.



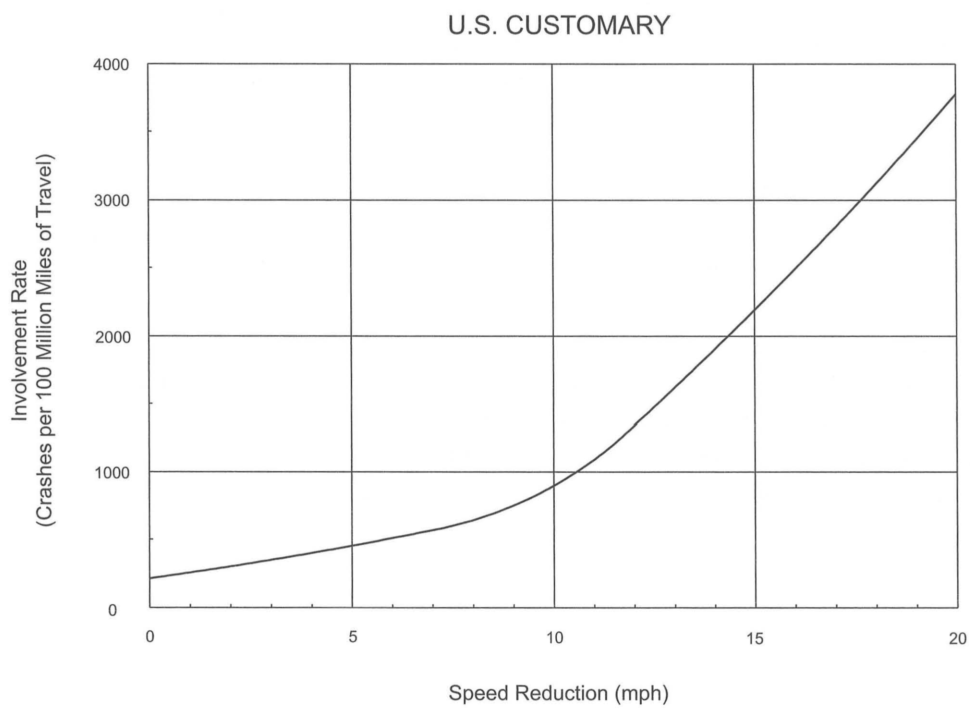


Figure 3-27. Crash Involvement Rate of Trucks for Which Running Speeds Are Reduced below Average Running Speed of All Traffic (*26*)

To establish design values for critical lengths of grade for which gradeability of trucks is the determining factor, data or assumptions are needed for the following:

1. Size and power of a representative truck or truck combination to be used as a design vehicle along with the gradeability data for this vehicle:

   Data show that the 85th percentile weight/power ratios for trucks on main highways are typically in the range from 102 to 126 kg/kW [170 to 210 lb/hp] (*33*). A typical loaded truck, powered so that the weight/power ratio is about 120 kg/kW [200 lb/hp], is representative of the size and type of vehicle normally used as a design control for main highways. Data in Figures 3-24 and 3-25 apply to such a vehicle. More powerful trucks with weight/power ratios in the range from 102 to 108 kg/kW [170 to 180 lb/hp] may be appropriate in some Western states, while some two-lane highways that are not major intercity routes may have distinctly different truck populations with weight/power ratios higher than 126 kg/kW [210 lb/hp].

2. Speed at entrance to critical length of grade:

   The average running speed as related to design speed can be used to approximate the speed of vehicles beginning an uphill climb. This estimate is, of course, subject to adjustment as approach conditions may determine. Where vehicles approach on nearly level grades, the running speed can be used directly. For a downhill approach it should be increased somewhat, and for an uphill approach it should be decreased.

3. Minimum speed on the grade below in which interference to following vehicles is considered unreasonable:

   No specific data are available on which to base minimum tolerable speeds of trucks on upgrades. It is logical to assume that such minimum speeds should be in direct relation to the design speed. Minimum truck speeds of about 40 to 60 km/h [25 to 40 mph] for the majority of highways (on which design speeds are about 60 to 100 km/h [40 to 60 mph]) probably are not unreasonably annoying to following drivers unable to pass on two-lane roads, if the time interval during which they are unable to pass is not too long. The time interval is less likely to be annoying on two lane roads with volumes well below their capacities, whereas it is more likely to be annoying on two-lane roads with volumes near capacity. Lower minimum truck speeds can probably be tolerated on multilane highways rather than on two-lane roads because there is more opportunity for and less difficulty in passing. Highways should be designed so that the speeds of trucks will not be reduced enough to cause intolerable conditions for following drivers.

Studies show that, regardless of the average speed on the highway, the more a vehicle deviates from the average speed, the greater its chances of becoming involved in a crash. One such study (*25*) used the speed distribution of vehicles traveling on highways in one state, and related it to the crash involvement rate to obtain the rate for trucks of four or more axles operating on level grades. The crash involvement rates for truck speed reductions of 10, 15, 25, and 30 km/h [5, 10, 15, and 20 mph] were developed assuming the reduction in the average speed for all vehicles on a grade was 30 percent of the truck speed reduction on the same grade. The results of this analysis are shown in Figure 3-27.

A common basis for determining critical length of grade is based on a reduction in speed of trucks below the average running speed of traffic. The ideal would be for all traffic to operate at the average speed.

This, however, is not practical. In the past, the general practice has been to use a reduction in truck speed of 25 km/h [15 mph] below the average running speed of all traffic to identify the critical length of grade. As shown in Figure 3-27, the crash involvement rate increases significantly when the truck speed reduction exceeds 15 km/h [10 mph] with the involvement rate being 2.4 times greater for a 25-km/h [15-mph] reduction than for a 15-km/h [10-mph] reduction. On the basis of these relationships, it is recommended that a 15-km/h [10 mph] reduction criterion be used as the general guide for determining critical lengths of grade.

The length of any given grade that will cause the speed of a representative truck (120 kg/kW [200 lb/hp]) entering the grade at 110 km/h [70 mph] to be reduced by various amounts below the average running speed of all traffic is shown graphically in Figure 3-28, which is based on the truck performance data presented in Figure 3-24. The curve showing a 15-km/h [10-mph] speed reduction is used as the general design guide for determining the critical lengths of grade. Similar information on the critical length of grade for recreational vehicles may be found in Figure 3-29, which is based on the recreational vehicle performance data presented in Figure 3-26.

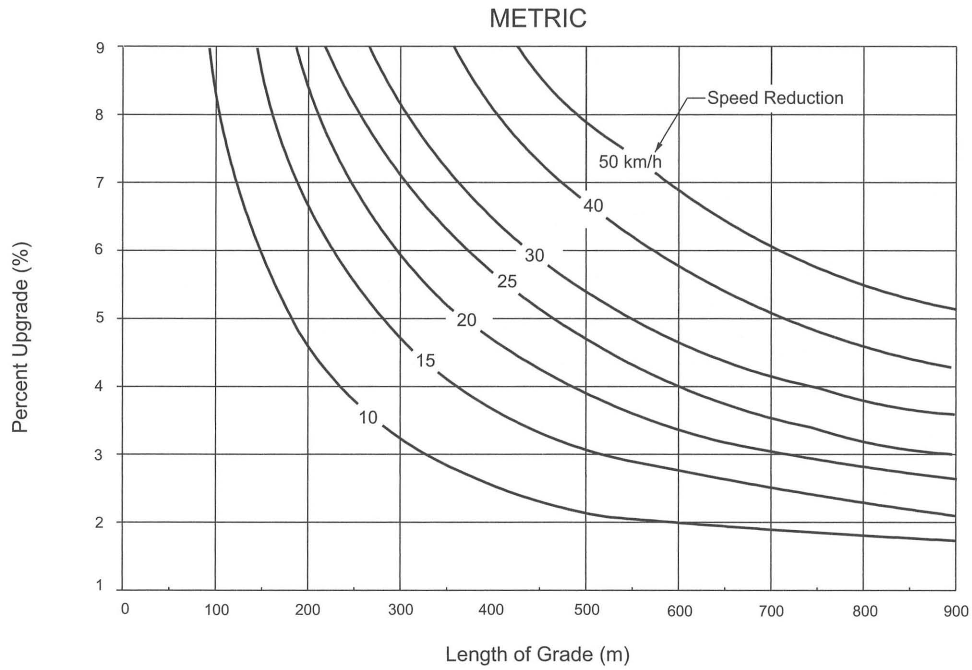
Where the entering speed is less than 110 km/h [70 mph], as may be the case where the approach is on an upgrade, the speed reductions shown in Figures 3-28 and 3-29 will occur over shorter lengths of grade. Conversely, where the approach is on a downgrade, the probable approach speed is greater than 110 km/h [70 mph] and the truck or recreational vehicle will ascend a greater length of grade than shown in the figures before the speed is reduced to the values shown.

The method of using Figure 3-28 to determine critical lengths of grade is demonstrated in the following examples.

Assume that a highway is being designed for 100 km/h [60 mph] and has a fairly level approach to a 4 percent upgrade. The 15-km/h [10-mph] speed reduction curve in Figure 3-28 shows the critical length of grade to be 350 m [1,200 ft]. If, instead, the design speed was 60 km/h [40 mph], the initial and minimum tolerable speeds on the grade would be different, but for the same permissible speed reduction the critical length would still be 350 m [1,200 ft].

In another instance, the critical length of a 5 percent upgrade approached by a 500-m [1,650-ft] length of 2 percent upgrade is unknown. Figure 3-28 shows that a 2 percent upgrade of 500 m [1,650 ft] in length would result in a speed reduction of about 9 km/h [6 mph]. The chart further shows that the remaining tolerable speed reduction of 6 km/h [4 mph] would occur on 100 m [325 ft] of the 5 percent upgrade.

Where an upgrade is approached on a downgrade, heavy trucks often increase speed considerably to begin the climb on the upgrade at as high a speed as practical. This factor can be recognized in design by increasing the tolerable speed reduction. It remains for the designer to judge to what extent the speed of trucks would increase at the bottom of the momentum grade above that generally found on level approaches. It appears that a speed increase of about 10 km/h [5 mph] can be considered for moderate downgrades and a speed increase of 15 km/h [10 mph] for steeper grades of moderate length or longer. On this basis, the tolerable speed reduction with momentum grades would be 25 or 30 km/h [15 or 20 mph]. For example, where there is a moderate length of 4 percent downgrade in advance of a 6 percent upgrade, a tolerable speed reduction of 25 km/h [15 mph] can be assumed. For this case, the critical length of the 6 percent upgrade is about 370 m [1,250 ft].



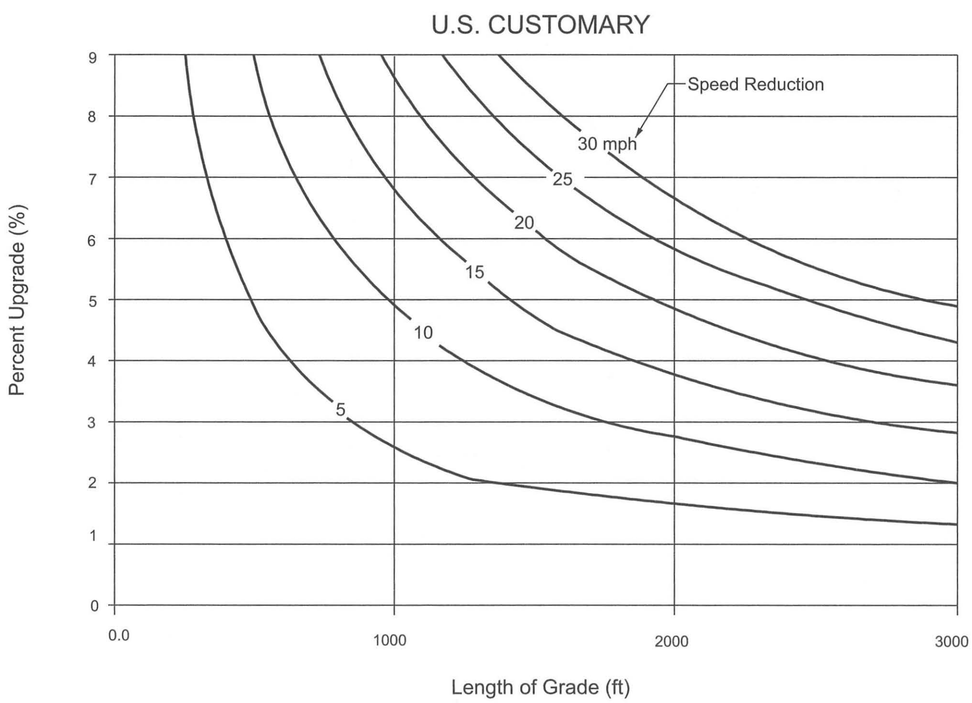


Figure 3-28. Critical Lengths of Grade for Design, Assumed Typical Heavy Truck of 120 kg/kW [200 lb/hp], Entering Speed = 110 km/h [70 mph]

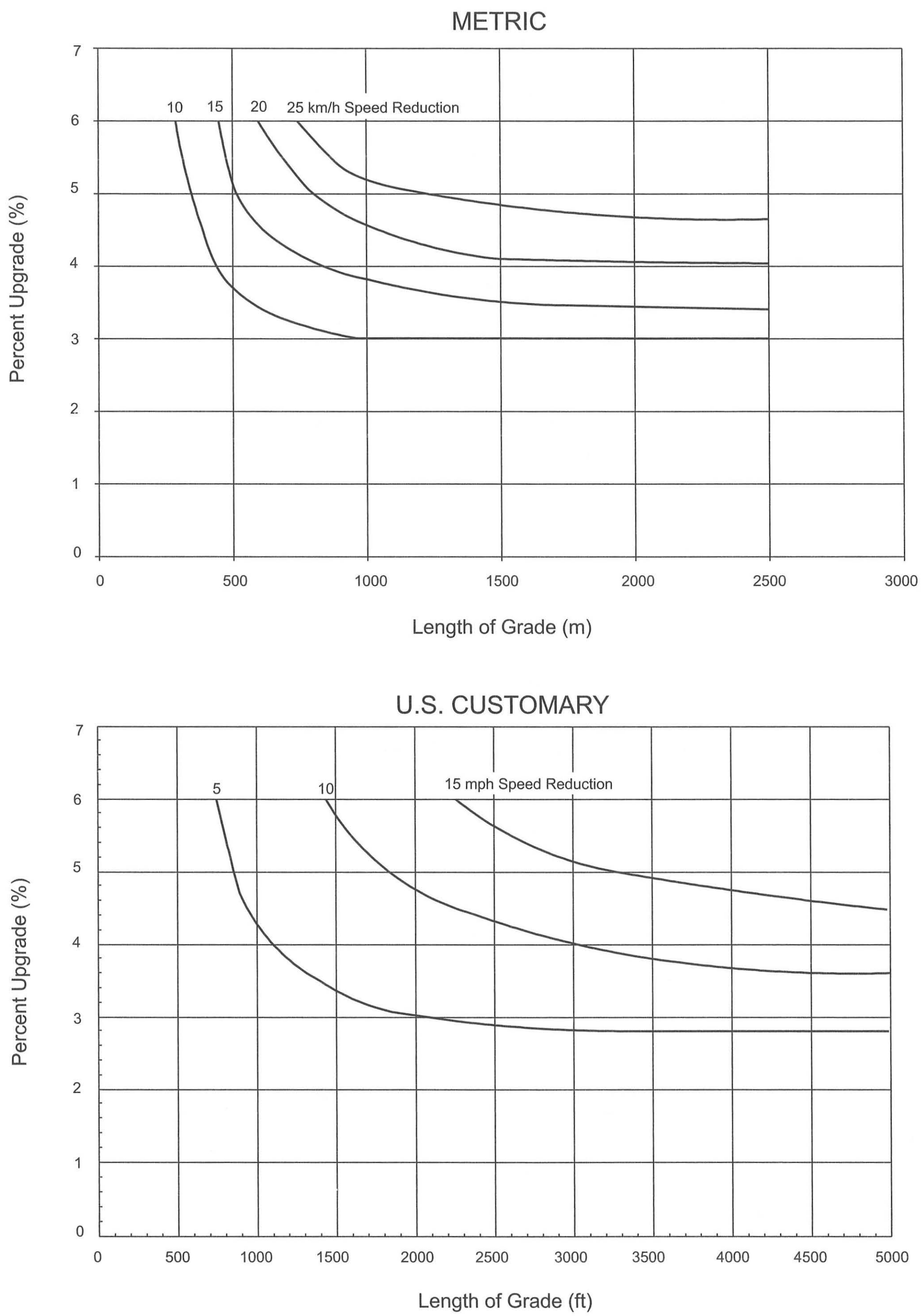


Figure 3-29. Critical Lengths of Grade Using an Approach Speed of 90 km/h [55 mph] for Typical Recreational Vehicle (*18*)

The critical length of grade in Figure 3-28 is derived as the length of tangent grade. Where a vertical curve is part of a critical length of grade, an approximate equivalent tangent grade length should be used. Where the condition involves vertical curves of Types II and IV shown in Section 3.4.6 in Figure 3-41 and the algebraic difference in grades is not too great, the measurement of critical length of grade may be made between the vertical points of intersection (VPI). Where vertical curves of Types I and III in Figure 3-41 are involved, about one-quarter of the vertical curve length should be regarded as part of the grade under consideration.

In many design situations, Figure 3-28 may not be directly applicable to the determination of the critical length of grade for one of several reasons. First, the truck population for a given site may be such that a weight/power ratio either less than or greater than the value of 120 kg/kW [200 lb/hp] assumed in Figure 3-28 may be appropriate as a design control. Second, for the reasons described above, the truck speed at the entrance to the grade may differ from the 110 km/h [70 mph] assumed in Figure 3-28. Third, the profile may not consist of a constant percent grade. In such situations, a spreadsheet program, known as the Truck Speed Profile Model (TSPM) (*33*), is available and may be used to generate truck speed profiles for any specified truck weight/power ratio, initial truck speed, and any sequence of grades.

Steep downhill grades on facilities with high traffic volumes and numerous heavy trucks can reduce the traffic capacity and increase crash frequency. Some downgrades are long and steep enough that some heavy vehicles travel at crawl speeds to avoid loss of control on the grade. Slow-moving vehicles of this type may impede other vehicles. Therefore, there are instances where consideration should be given to providing a truck lane for downhill traffic. Procedures have been developed in the HCM (*62*) to analyze this situation.

The suggested design criterion for determining the critical length of grade is not intended as a strict control but as a guideline. In some instances, the terrain or other physical controls may preclude shortening or flattening grades to meet these controls. Where a speed reduction greater than the suggested design guide cannot be avoided, undesirable operation may result on roads with numerous trucks, particularly on two-lane roads with volumes approaching capacity and in some instances on multilane highways. Where the length of critical grade is exceeded, consideration should be given to providing an added uphill lane for slow-moving vehicles, particularly where volume is at or near capacity and the truck volume is high. Data in Figure 3-28 can be used along with other key factors, particularly volume data in relation to capacity and volume data for trucks, to determine where such added lanes are warranted.

3.4.3 Climbing Lanes

Climbing Lanes for Two-Lane Highways

General—Freedom and safety of operation on two-lane highways, besides being influenced by the extent and frequency of passing sections, are adversely affected by heavily loaded vehicle traffic operating on grades of sufficient length to result in speeds that could impede following vehicles. In the past, provision of added climbing lanes to improve operations on upgrades has been rather limited because of the additional construction costs. However, because of the increasing amount of delay and the number of serious crashes occurring on grades, such lanes are now more commonly included in original construction plans, and additional lanes on existing highways are being considered as safety improvement projects. The crash potential created by this condition is illustrated in Figure 3-27.

A highway section with a climbing lane is not considered a three-lane highway, but a two-lane highway with an added lane for vehicles moving slowly uphill so that other vehicles using the normal lane to the right of the centerline are not delayed. These faster vehicles pass the slower vehicles moving upgrade, but not in the lane for opposing traffic, as on a conventional two-lane road. A separate climbing lane exclusively for slow-moving vehicles is preferred to the addition of an extra lane carrying mixed traffic. Designs of two-lane highways with climbing lanes are illustrated in Figures 3-30A and 3-30B. Climbing lanes are designed for each direction independently of the other. Depending on the alignment and profile conditions, they may not overlap, as in Figure 3-30A, or they may overlap, as in Figure 3-30B, where there is a crest with a long grade on each side.

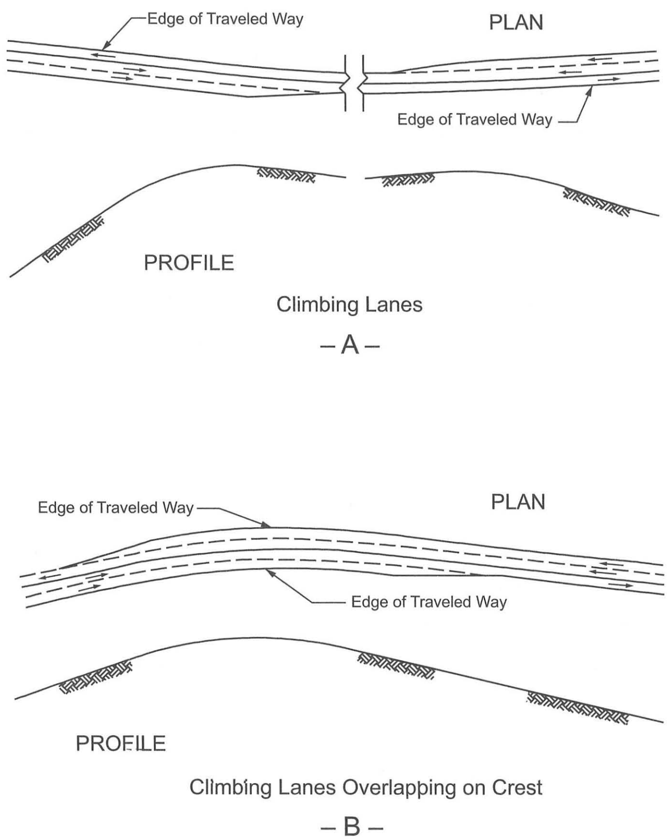


Figure 3-30. Climbing Lanes on Two-Lane Highways

Adding a climbing lane for an upgrade on a two-lane highway can offset the decline in traffic operations caused by the combined effects of the grade, traffic volume, and heavy vehicles. Climbing lanes are appropriate where the level of service or the speed of trucks is substantially less on an upgrade than on the approach to the upgrade. Where climbing lanes are provided, there has been a high degree of compliance in their use by truck drivers.

On highways with low volumes, only an occasional car is delayed, and climbing lanes, although desirable, may not be justified economically even where the critical length of grade is exceeded. For such cases, slow-moving vehicle turnouts should be considered to reduce delay to occasional passenger cars from slow-moving vehicles. Turnouts are discussed in Section 3.4.4, "Methods for Increasing Passing Opportunities on Two-Lane Roads."

The following three criteria, reflecting economic considerations, should be satisfied to justify a climbing lane:

1. Upgrade traffic flow rate in excess of 200 vehicles per hour.

2. Upgrade truck flow rate in excess of 20 vehicles per hour.

3. One of the following conditions exists:

   - A 15-km/h [10-mph] 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.

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 15-km/h [10-mph] speed reduction for trucks is found using Figure 3-28. 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 (*62*) 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 15-km/h [10-mph] 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 (*62*) 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 60 km/h [40 mph], but a 15-km/h [10-mph] decrease in truck speed below the average running speed, as discussed in "Critical Lengths of Grade for Design" of Section 3.4.2, is the most practical reduction obtainable from the standpoint of level of service and crash frequency. This 15-km/h [10-mph] 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 15 km/h [10 mph] below the average running speed may be determined from Figures 3-24 or 3-28. Different curves would apply for trucks with other than a weight/power ratio of 120 kg/kW [200 lb/hp]. For example, assuming an approach condition on which trucks with a 120-kg/kW [200-lb/hp] weight/power ratio are traveling within a flow having an average running speed of 110 km/h [70 mph], the resulting 15-km/h [10-mph] speed reduction occurs at distances of approximately 175 to 350 m [600 to 1,200 ft] 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 likelihood of low speeds for approaching trucks, the added lane should be introduced near the foot of the grade. The beginning of the added lane should be preceded by a tapered section with a desirable taper ratio of 25:1 that should be at least 90 m [300 ft] long.

The ideal design is to extend a climbing lane to a point beyond the crest, where a typical truck could attain a speed that is within 15 km/h [10 mph] of the speed of the other vehicles with a desirable speed of at least 60 km/h [40 mph]. This may not be practical in many instances because of the unduly long distance needed for trucks to accelerate to the desired speed. In such situations, a practical point to end the added lane is where trucks can return to the normal lane without undue interference with other traffic—in particular, where the sight distance becomes sufficient to permit passing when there is no oncoming traffic or, preferably, at least 60 m [200 ft] beyond that point. An appropriate taper length should be provided to permit trucks to return smoothly to the normal lane. For example, on a highway where the passing sight distance becomes available 30 m [100 ft] beyond the crest of the grade, the climbing lane should extend

90 m [300 ft] beyond the crest (i.e., 30 m [100 ft] plus 60 m [200 ft]), and an additional tapered section with a desirable taper ratio of 50:1 that should be at least 180 m [600 ft] long.

A climbing lane should desirably be as wide as the through lanes. It should be so constructed that it can immediately be recognized as an added lane for one direction of travel. The centerline of the normal two-lane highway should be clearly marked, including yellow barrier lines for no passing zones. Signs at the beginning of the upgrade such as "Slower Traffic Keep Right" or "Trucks Use Right Lane" may be used to direct slow-moving vehicles into the climbing lane. These and other appropriate signs and markings for climbing lanes are presented in the MUTCD (*22*).

The cross slope of a climbing lane is usually handled in the same manner as the addition of a lane to a multilane highway. Depending on agency practice, this design results in either a continuation of the cross slope or a lane with slightly more cross slope than the adjacent through lane. On a superelevated section, the cross slope is generally a continuation of the slope used on the through lane.

Desirably, the shoulder on the outer edge of a climbing lane should be as wide as the shoulder on the normal two-lane cross section, particularly where there is bicycle traffic. However, this may be impractical, particularly when the climbing lane is added to an existing highway. A usable shoulder of 1.2 m [4 ft] in width or greater is acceptable. Although not wide enough for a stalled vehicle to completely clear the climbing lane, a 1.2-m [4-ft] shoulder in combination with the climbing lane generally provides sufficient width for both the stalled vehicle and a slow-speed passing vehicle without need for the latter to encroach on the through lane.

In summary, climbing lanes offer a comparatively inexpensive means of overcoming reductions in capacity and providing improved operation where congestion on grades is caused by slow trucks in combination with high traffic volumes. As discussed earlier in this section, climbing lanes also reduce crashes. On some existing two-lane highways, the addition of climbing lanes could defer reconstruction for many years or indefinitely. In a new design, climbing lanes could make a two-lane highway operate efficiently, whereas a much more costly multilane highway would be needed without them.

Climbing Lanes on Freeways and Multilane Highways

General—Climbing lanes, although they are becoming more prevalent, have not been used as extensively on freeways and multilane highways as on two-lane highways. This may result from multilane facilities more frequently having sufficient capacity to serve their traffic demands, including the typical percentage of slow-moving vehicles with high weight/power ratios, without being congested. Climbing lanes are generally not as easily justified on multilane facilities as on two-lane highways, because on two-lane facilities vehicles following other slower moving vehicles on upgrades are frequently prevented from passing in the adjacent traffic lane by opposing traffic. On multilane facilities, there is no such impediment to passing. A slow-moving vehicle in the normal right lane does not impede the following vehicles that can readily move left to the adjacent lane and proceed without difficulty, although there is evidence that crashes are reduced when vehicles in the traffic stream move at the same speed.

Because highways are normally designed for 20 years or more in the future, there is less likelihood that climbing lanes will be justified on multilane facilities than on two-lane roads for several years after construction even though they are deemed desirable for the peak hours of the design year. Where this is the case, there is economic advantage in designing for, but deferring construction of, climbing lanes on multilane facilities. In this situation, grading for the future climbing lane should be provided initially. The additional grading needed for a climbing lane is small when compared to that needed for the overall cross section. If, however, even this additional grading is impractical, it is acceptable, although not desirable, to use a narrower shoulder adjacent to the climbing lane rather than the full shoulder provided on a normal section.

Although primarily applicable in rural areas, there are instances where climbing lanes are needed in urban areas. Climbing lanes are particularly important for freedom of operation on urban freeways where traffic volumes are high in relation to capacity. On older urban freeways and arterial streets with appreciable grades and no climbing lanes, it is a common occurrence for heavy traffic, which may otherwise operate well, to platoon on grades.

Trucks—The principal determinants of the need for climbing lanes on multilane highways are critical lengths of grade, effects of trucks on grades in terms of equivalent passenger car flow rates, and service volumes for the desired level of service and the next poorer level of service.

Critical length of grade has been discussed previously in Section 3.4.2. It is the length of a particular upgrade that reduces the speed of low-performance trucks 15 km/h [10 mph] below the average running speed of the remaining traffic. The critical length of grade that results in a 15 km/h [10-mph] reduction of truck speed is found using Figure 3-28 and is then compared to the length of the particular grade being examined. If the critical length of grade is less than the length of grade being evaluated, consideration of a climbing lane is warranted.

In determining service volume, the passenger-car equivalent for trucks is a significant factor. It is generally agreed that trucks on multilane facilities have less effect in deterring following vehicles than on two-lane roads. Comparison of passenger-car equivalents in the HCM (*62*) for the same percent of grade, length of grade, and percent of trucks clearly illustrates the difference in passenger-car equivalents of trucks for two-lane and multilane facilities.

To justify the cost of providing a climbing lane, the existence of a low level of service on the grade should be the criterion, as in the case of justifying climbing lanes for two-lane roads, because highway users will accept a higher degree of congestion (i.e., a lower level of service) on individual grades than over long sections of highway. As a matter of practice, the service volume on an individual grade should not exceed that for the next poorer level of service from that used for the basic design. The one exception is that the service volume for level of service D should not be exceeded.

Generally, climbing lanes should not be considered unless the directional traffic volume for the upgrade is equal to or greater than the service volume for level of service D. In most cases when the service volume, including trucks, is greater than 1,700 vehicles per hour per lane and the length of the grade and the percentage of trucks are sufficient to consider climbing lanes, the volume in terms of equivalent passenger cars is likely to approach or even exceed the capacity. In this situation, an increase in the number of lanes throughout the highway section would represent a better investment than the provision of climbing lanes.

Climbing lanes are also not generally warranted on four-lane highways with directional volumes below 1,000 vehicles per hour per lane regardless of the percentage of trucks. Although a truck driver will occasionally pass another truck under such conditions, the inconvenience with this low volume is not sufficient to justify the cost of a climbing lane in the absence of appropriate criteria.

The procedures in the HCM (*62*) should be used to consider the traffic operational characteristics on the grade being examined. The maximum service flow rate for the desired level of service, together with the flow rate for the next poorer level of service, should be determined. If the flow rate on the grade exceeds the service flow rate of the next poorer level of service, consideration of a climbing lane is warranted. In order to use the HCM procedures, the free-flow speed should be determined or estimated. The free-flow speed can be determined by measuring the mean speed of passenger cars under low to moderate flow conditions (up to 1,300 passenger cars per hour per lane) on the facility or similar facility.

Data (*25*, *62*) indicate that the mean free-flow speed under ideal conditions for multilane highways ranges from 0.6 km/h [1 mph] lower than the 85th percentile speed of 65 km/h [40 mph] to 5 km/h [3 mph] lower than the 85th percentile speed of 100 km/h [60 mph]. Speed limit is one factor that affects free-flow speed. Research (*25*, *62*) suggests that the free-flow speed is approximately 11 km/h [7 mph] higher than the speed limit on facilities with 65- and 70-km/h [40- and 45-mph] speed limits and 8 km/h [5 mph] higher than the speed limit on facilities with 80- and 90-km/h [50- and 55-mph] speed limits. Analysis based on these rules of thumb should be used with caution. Field measurement is the recommended method of determining the free-flow speed, with estimation using the above procedures employed only when field data are not available.

Where the grade being investigated is located on a multilane highway, other factors should sometimes be considered; such factors include median type, lane widths, lateral clearance, and access point density. These factors are accounted for in the capacity analysis procedures by making adjustments in the free-flow speed and are not normally a separate consideration in determining whether a climbing lane would be advantageous.

For freeways, adjustments are made in traffic operational analyses using factors for restricted lane widths, lateral clearances, recreational vehicles, and unfamiliar driver populations. The HCM (*62*) should be used for information on considering these factors in analysis.

Under certain circumstances there should be consideration of additional lanes to accommodate trucks in the downgrade direction. This is accomplished using the same procedure as described above and using the passenger-car equivalents for trucks on downgrades in place of the values for trucks and recreational vehicles on upgrades.

Climbing lanes on multilane roads are usually placed on the outer or right-hand side of the roadway as shown in Figure 3-31. The principles for cross slopes, for locating terminal points, and for designing terminal areas or tapers for climbing lanes are discussed in the earlier portion of this section on "Climbing Lanes for Two-Lane Highways"; these principles are equally applicable to climbing lanes on multilane facilities. A primary consideration is that the location of the uphill terminus of the climbing lane should be at the point where a satisfactory speed is attained by trucks, preferably about 15 km/h [10 mph] below the average running speed of the highway. Passing sight distance need not be considered on multilane highways.

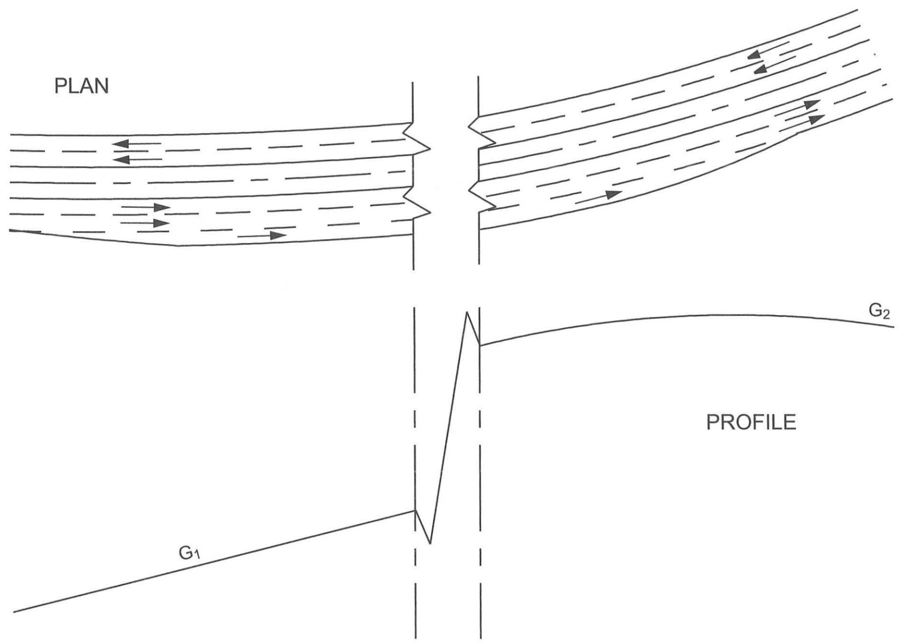


Figure 3-31. Climbing Lane on Freeways and Multilane Highways

3.4.4 Methods for Increasing Passing Opportunities on Two-Lane Roads

Several highway agencies have pioneered successful methods for providing more passing opportunities along two-lane roads. Some of the more recognized of these methods, including passing lanes, turnouts, shoulder driving, and shoulder use sections are described in the FHWA informational guide *Low Cost Methods for Improving Traffic Operations on Two-Lane Roads* (*29*). An additional design alternative or method known as a 2+1 roadway has been reported in *NCHRP Research Results Digest 275, Application of European 2+1 Roadway Designs* (*49*). A synopsis of portions of material found in these sources is presented in the remainder of this section. More detailed criteria for these methods are found in the referenced documents.

Passing Lanes

An added lane can be provided in one or both directions of travel to improve traffic operations in sections of lower capacity to at least the same quality of service as adjacent road sections. Passing lanes can also be provided to improve overall traffic operations on two-lane highways by reducing delays caused by inadequate passing opportunities over significant lengths of highways, typically 10 to 100 km [6 to 60 miles]. Where passing lanes are used to improve traffic operations over a length of road, they frequently are provided systematically at regular intervals.

The location of the added lane should appear logical to the driver. The value of a passing lane is more obvious at locations where passing sight distance is restricted than on long tangents that may provide passing opportunities even without passing lanes. On the other hand, the location of a passing lane should recognize the need for adequate sight distance at both the lane addition and lane drop tapers. A minimum sight distance of 300 m [1,000 ft] on the approach to each taper is recommended. The selection of an appropriate location also needs to consider the location of intersections and high-volume driveways in order to minimize the volume of turning movements on a road section where passing is encouraged. Furthermore, other physical constraints such as bridges and culverts should be avoided if they restrict provision of a continuous shoulder.

The following is a summary of the design procedure to be followed in providing passing sections on two-lane highways:

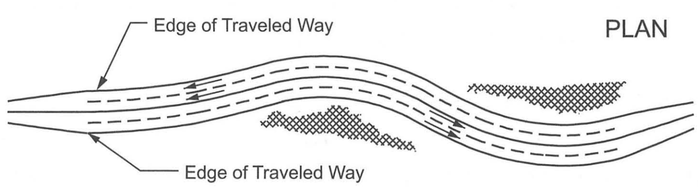
1. Horizontal and vertical alignment should be designed to provide as much of the highway as practical with passing sight distance (see Table 3-4).

2. Where the design volume approaches capacity, the effect of lack of passing opportunities in reducing the level of service should be recognized.

3. Where the critical length of grade is less than the physical length of an upgrade, consideration should be given to constructing added climbing lanes. The critical length of grade is determined as shown in Figures 3-28 and 3-29.

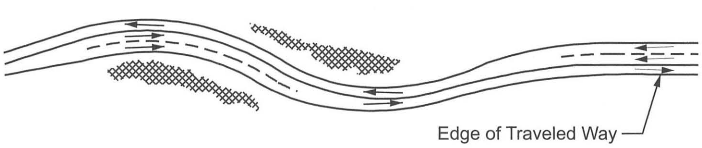
4. Where the extent and frequency of passing opportunities made available by application of Criteria 1 and 3 are still too few, consideration should be given to the construction of passing-lane sections.

Passing-lane sections, which may be either three or four lanes in width, are constructed on two-lane roads to provide the desired frequency of passing zones or to eliminate interference from low-speed heavy vehicles, or both. Where a sufficient number and length of passing sections cannot be obtained in the design of horizontal and vertical alignment alone, an occasional added lane in one or both directions of travel may be introduced as shown in Figure 3-32 to provide more passing opportunities. Such sections are particularly advantageous in rolling terrain, especially where alignment is winding or the profile includes critical lengths of grade.



Four-Lane Passing Section on Two-Lane Highway

– A –

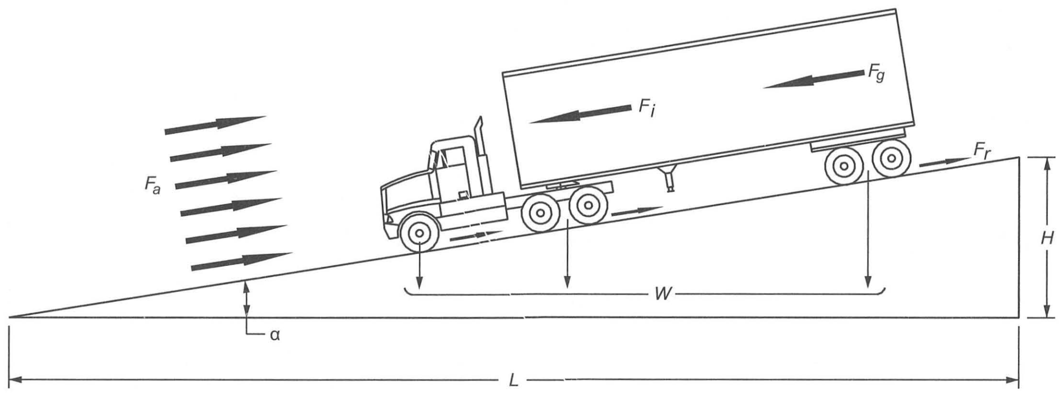


Three-Lane Passing Section on Two-Lane Highway

– B –

Figure 3-32. Passing Lanes Section on Two-Lane Roads

and gradient. Inertial and negative gradient forces act to maintain motion of the vehicle, while rolling-, positive gradient-, and air-resistance forces act to retard its motion. Figure 3-38 illustrates the action of the various resistance forces on a vehicle.



F_a = Air resistance
F_i = Inertial resistance
F_g = Gradient resistance
F_r = Rolling resistance

W = Gross vehicle mass [weight]
H = Height
L = Length
α = Slope Angle

Figure 3-38. Forces Acting on a Vehicle in Motion

Inertial resistance can be described as a force that resists movement of a vehicle at rest or maintains a vehicle in motion, unless the vehicle is acted on by some external force. Inertial resistance must be overcome to either increase or decrease the speed of a vehicle. Rolling- and positive gradient-resistance forces are available to overcome the inertial resistance. Rolling resistance is a general term used to describe the resistance to motion at the area of contact between a vehicle's tires and the roadway surface and is only applicable when a vehicle is in motion. It is influenced by the type and displacement characteristics of the surfacing material of the roadway. Each surfacing material has a coefficient, expressed in kg/1 000 kg [lb/1,000 lb] of gross vehicle weight (GVM [GVW]), which determines the amount of rolling resistance of a vehicle. The values shown in Table 3-33 for rolling resistance have been obtained from various sources throughout the country and are a best available estimate.

Gradient resistance results from gravity and is expressed as the force needed to move the vehicle through a given vertical distance. For gradient resistance to provide a beneficial force on an escape ramp, the vehicle must be moving upgrade, against gravity. In the case where the vehicle is descending a grade, gradient resistance is negative, thereby reducing the forces available to slow and stop the vehicle. The amount of gradient resistance is influenced by the total weight of the vehicle and the magnitude of the grade. For each percent of grade, the gradient resistance is 10 kg/1 000 kg [10 lb/1,000 lb] whether the grade is positive or negative.

The remaining component of tractive resistance is aerodynamic resistance, the force resulting from the retarding effect of air on the various surfaces of the vehicle. Air causes a significant resistance at speeds above 80 km/h [50 mph], but is negligible under 30 km/h [20 mph]. The effect of aerodynamic resistance has been neglected in determining the length of the arrester bed, thus providing a small additional margin of safety.

Table 3-33. Rolling Resistance of Roadway Surfacing Materials

	Metric		U.S. Customary	
Surfacing Material	Rolling Resistance (kg/1 000 kg GVM)	Equivalent Grade (%)[a]	Rolling Resistance (lb/1,000 lb GVW)	Equivalent Grade (%)[a]
Portland cement concrete	10	1.0	10	1.0
Asphalt concrete	12	1.2	12	1.2
Gravel, compacted	15	1.5	15	1.5
Earth, sandy, loose	37	3.7	37	3.7
Crushed aggregate, loose	50	5.0	50	5.0
Gravel, loose	100	10.0	100	10.0
Sand	150	15.0	150	15.0
Pea gravel	250		250	25.0

[a] Rolling resistance expressed as equivalent gradient.

Need and Location for Emergency Escape Ramps

Each grade has its own unique characteristics. Highway alignment, gradient, length, and descent speed contribute to the potential for out-of-control vehicles. For existing highways, operational concerns on a downgrade will often be reported by law enforcement officials, truck drivers, or the general public. A field review of a specific grade may reveal damaged guardrail, gouged pavement surfaces, or spilled oil indicating locations where drivers of heavy vehicles had difficulty negotiating a downgrade. For existing facilities, an escape ramp should be provided as soon as a need is established. Crash experience (or, for new facilities, crash experience on similar facilities) and truck operations on the grade combined with engineering judgment are frequently used to determine the need for a truck escape ramp. Often the impact of a potential runaway truck on adjacent activities or population centers will provide sufficient reason to construct an escape ramp.

Unnecessary escape ramps should be avoided. For example, a second escape ramp should not be needed just beyond the curve that created the need for the initial ramp.

While there are no universal guidelines available for new and existing facilities, a variety of factors should be considered in selecting the specific site for an escape ramp. Each location presents a different array of design needs; factors that should be considered include topography, length and percent of grade, potential speed, economics, environmental impact, and crash experience. Ramps should be located to intercept the greatest number of runaway vehicles, such as at the bottom of the grade and at intermediate points along the grade where an out-of-control vehicle could cause a catastrophic crash.

A technique for new and existing facilities available for use in analyzing operations on a grade, in addition to crash analysis, is the *Grade Severity Rating System* (*19*). The system uses a predetermined brake temperature limit (260°C [500°F]) to establish a safe descent speed for the grade. It also can be used to determine expected brake temperatures at 0.8-km [0.5-mi] intervals along the downgrade. The location where brake temperatures exceed the limit indicates the point that brake failures can occur, leading to potential runaways.

Escape ramps generally may be built at any practical location where the main road alignment is tangent. They should be built in advance of horizontal curves that cannot be negotiated safely by an out-of-control vehicle without rolling over and in advance of populated areas. Escape ramps should exit to the right of the roadway. On divided multilane highways, where a left exit may appear to be the only practical location, difficulties may be expected by the refusal of vehicles in the left lane to yield to out-of-control vehicles attempting to change lanes.

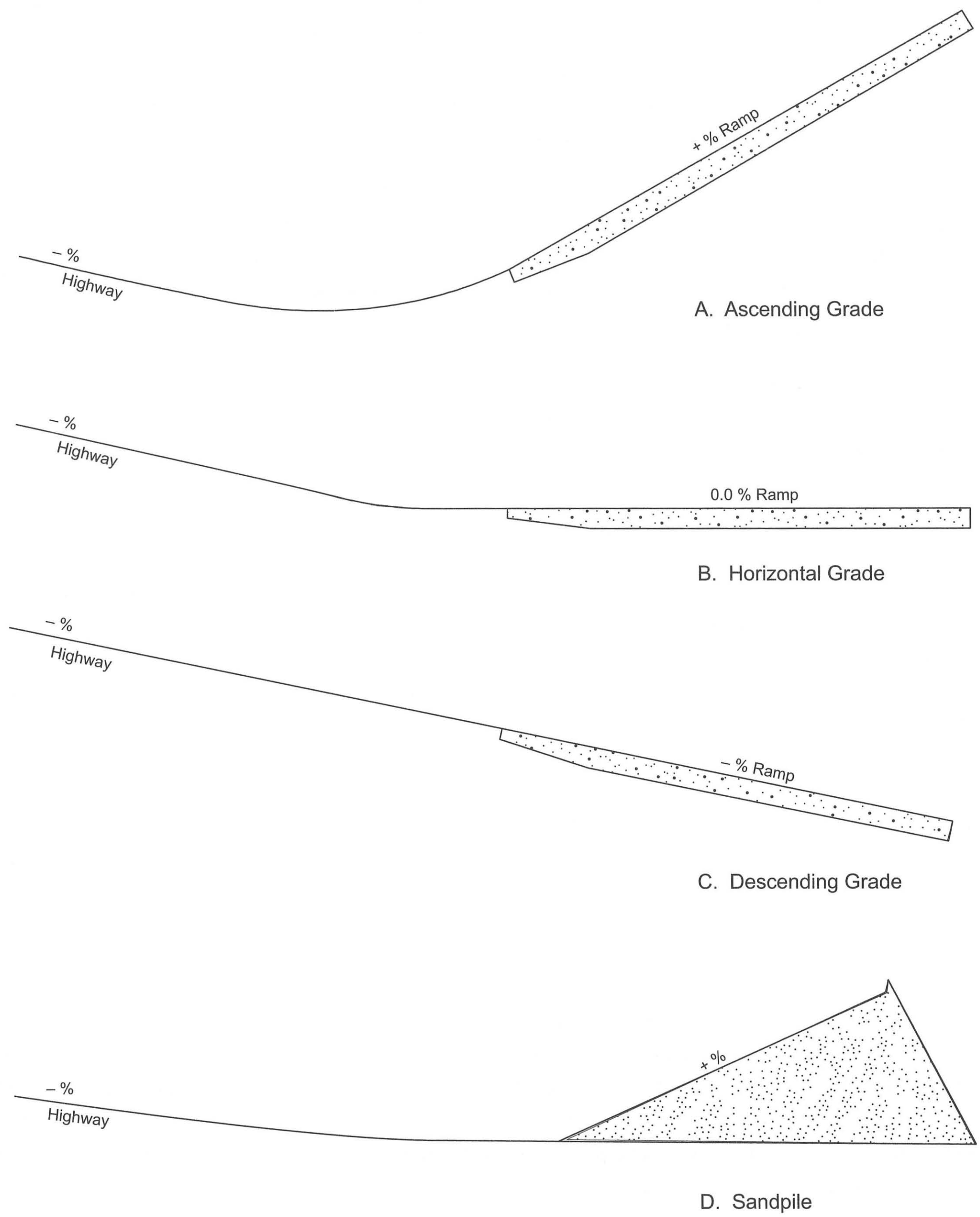
Although crashes involving runaway trucks can occur at various sites along a grade, locations having multiple crashes should be analyzed in detail. Analysis of crash data pertinent to a prospective escape ramp site should include evaluation of the section of highway immediately uphill, including the amount of curvature traversed and distance to and radius of the adjacent curve.

An integral part of the evaluation should be the determination of the maximum speed that an out-of-control vehicle could attain at the proposed site. This highest obtainable speed can then be used as the minimum design speed for the ramp. The 130- to 140-km/h [80- to 90-mph] entering speed, recommended for design, is intended to represent an extreme condition and therefore should not be used as the basis for selecting locations of escape ramps. Although the variables involved make it impractical to establish a maximum truck speed warrant for location of escape ramps, it is evident that anticipated speeds should be below the range used for design. The principal factor in determining the need for an emergency escape ramp should be the safety of the other traffic on the roadway, the driver of the out-of-control vehicle, and the residents along and at the bottom of the grade. An escape ramp, or ramps if the conditions indicate the need for more than one, should be located wherever grades are of a steepness and length that present a substantial risk of runaway trucks and topographic conditions will permit construction.

Types of Emergency Escape Ramps

Emergency escape ramps have been classified in a variety of ways. Three broad categories used to classify ramps are gravity, sandpile, and arrester bed. Within these broad categories, four basic emergency escape ramp designs predominate. These designs are the sandpile and three types of arrester beds, classified by grade of the arrester bed: descending grade, horizontal grade, and ascending grade. These four types are illustrated in Figure 3-39.

The gravity ramp has a paved or densely compacted aggregate surface, relying primarily on gravitational forces to slow and stop the runaway. Rolling-resistance forces contribute little to assist in stopping the vehicle. Gravity ramps are usually long, steep, and are constrained by topographic controls and costs. While a gravity ramp stops forward motion, the paved surface cannot prevent the vehicle from rolling back down the ramp grade and jackknifing without a positive capture mechanism. Therefore, the gravity ramp is the least desirable of the escape ramp types.



Note: Profile is along the baseline of the ramp.

Figure 3-39. Basic Types of Emergency Escape Ramps

Sandpiles, composed of loose, dry sand dumped at the ramp site, are usually no more than 120 m [400 ft] in length. The influence of gravity is dependent on the slope of the surface. The increase in rolling resistance is supplied by loose sand. Deceleration characteristics of sandpiles are usually severe and the sand can be affected by weather. Because of the deceleration characteristics, the sandpile is less desirable than the arrester bed. However, at locations where inadequate space exists for another type of ramp, the sand-pile may be appropriate because of its compact dimensions.

Descending-grade arrester-bed escape ramps are constructed parallel and adjacent to the through lanes of the highway. These ramps use loose aggregate in an arrester bed to increase rolling resistance to slow the vehicle. The gradient resistance acts in the direction of vehicle movement. As a result, the descending-grade ramps can be rather lengthy because the gravitational effect is not acting to help reduce the speed of the vehicle. The ramp should have a clear, obvious return path to the highway so drivers who doubt the effectiveness of the ramp will feel they will be able to return to the highway at a reduced speed.

Where the topography can accommodate, a horizontal-grade arrester-bed escape ramp is another option. Constructed on an essentially flat gradient, the horizontal-grade ramp relies on the increased rolling resistance from the loose aggregate in an arrester bed to slow and stop the out-of-control vehicle, since the effect of gravity is minimal. This type of ramp is longer than the ascending-grade arrester bed.

The most commonly used escape ramp is the ascending-grade arrester bed. Ramp installations of this type use gradient resistance to advantage, supplementing the effects of the aggregate in the arrester bed, and generally, reducing the length of ramp needed to stop the vehicle. The loose material in the arresting bed increases the rolling resistance, as in the other types of ramps, while the gradient resistance acts in a downgrade direction, opposite to the direction of vehicle movement. The loose bedding material also serves to hold the vehicle in place on the ramp grade after it has come to a safe stop.

Each of the ramp types is applicable to a particular situation where an emergency escape ramp is desirable and should be compatible with established location and topographic controls at possible sites. The procedures used for analysis of truck escape ramps are essentially the same for each of the categories or types identified. The rolling-resistance factor for the surfacing material used in determining the length needed to slow and stop the runaway truck safely is the difference in the procedures.

Design Considerations

The combination of the above external resistance and numerous internal resistance forces not discussed acts to limit the maximum speed of an out-of-control vehicle. Speeds in excess of 130 to 140 km/h [80 to 90 mph] will rarely, if ever, be attained. Therefore, an escape ramp should be designed for a minimum entering speed of 130 km/h [80 mph], with a 140-km/h [90-mph] design speed being preferred. Several formulas and software programs have been developed to determine the runaway speed at any point on the grade. These methods can be used to establish a design speed for specific grades and horizontal alignments (*19*, *38*, *68*).

The design and construction of effective escape ramps involve a number of considerations as follows:

- To safely stop an out-of-control vehicle, the length of the ramp should be sufficient to dissipate the kinetic energy of the moving vehicle.
- The alignment of the escape ramp should be tangent or on very flat curvature to minimize the driver's difficulty in controlling the vehicle.
- The width of the ramp should be adequate to accommodate more than one vehicle because it is not uncommon for two or more vehicles to have need of the escape ramp within a short time. A minimum width of 8 m [26 ft] may be all that is practical in some areas, though greater widths are preferred.

Desirably, a width of 9 to 12 m [30 to 40 ft] would more adequately accommodate two or more out-of-control vehicles. Ramp widths less than indicated above have been used successfully in some locations where it was determined that a wider width was unreasonably costly or not needed. Widths of ramps in use range from 3.6 to 12 m [12 to 40 ft].

- The surfacing material used in the arrester bed should be clean, not easily compacted, and have a high coefficient of rolling resistance. When aggregate is used, it should be rounded, uncrushed, predominantly a single size, and as free from fine-size material as practical. Such material will maximize the percentage of voids, thereby providing optimum drainage and minimizing interlocking and compaction. A material with a low shear strength is desirable to permit penetration of the tires. The durability of the aggregate should be evaluated using an appropriate crush test. Pea gravel is representative of the material used most frequently, although loose gravel and sand are also used. A gradation with a top size of 40 mm [1.5 in.] has been used with success in several states. Material conforming to the AASHTO gradation No. 57 is effective if the fine-sized material is removed.

- Arrester beds should be constructed with a minimum aggregate depth of 1 m [3 ft]. Contamination of the bed material can reduce the effectiveness of the arrester bed by creating a hard surface layer up to 300 mm [12 in.] thick at the bottom of the bed. Therefore, an aggregate depth up to 1 100 mm [42 in.] is recommended. As the vehicle enters the arrester bed, the wheels of the vehicle displace the surface, sinking into the bed material, thus increasing the rolling resistance. To assist in decelerating the vehicle smoothly, the depth of the bed should be tapered from a minimum of 75 mm [3 in.] at the entry point to the full depth of aggregate in the initial 30 to 60 m [100 to 200 ft] of the bed.

- A positive means of draining the arrester bed should be provided to help protect the bed from freezing and avoid contamination of the arrester bed material. This can be accomplished by grading the base to drain, intercepting water prior to entering the bed, underdrain systems with transverse outlets, or edge drains. Geotextiles or paving can be used between the subbase and the bed materials to prevent infiltration of fine materials that may trap water. Where toxic contamination from diesel fuel or other material spillage is a concern, the base of the arrester bed may be paved with concrete and holding tanks to retain the spilled contaminants may be provided.

- The entrance to the ramp should be designed so that a vehicle traveling at a high rate of speed can enter safely. As much sight distance as practical should be provided preceding the ramp so that a driver can enter safely. The full length of the ramp should be visible to the driver. The angle of departure for the ramp should be small, usually 5 degrees or less. An auxiliary lane may be appropriate to assist the driver to prepare to enter the escape ramp. The main roadway surface should be extended to a point at or beyond the exit gore so that both front wheels of the out-of-control vehicle will enter the arrester bed simultaneously; this also provides preparation time for the driver before actual deceleration begins. The arrester bed should be offset laterally from the through lanes by an amount sufficient to preclude loose material being thrown onto the through lanes.

- Access to the ramp should be clearly indicated by exit signing to allow the driver of an out-of-control vehicle time to react, to minimize the possibility of missing the ramp. Advance signing is needed to inform drivers of the existence of an escape ramp and to prepare drivers well in advance of the decision point so that they will have enough time to decide whether or not to use the escape ramp. Regulatory signs near the entrance should be used to discourage other motorists from entering, stopping, or parking at or on the ramp. The path of the ramp should be delineated to define ramp edges and provide nighttime direction; for more information, see the MUTCD (*22*). Illumination of the approach and ramp is desirable.

- The characteristic that makes a truck escape ramp effective also makes it difficult to retrieve a vehicle captured by the ramp. A service road located adjacent to the arrester bed is needed so tow trucks and maintenance vehicles can use it without becoming trapped in the bedding material. The width of this service road should be at least 3 m [10 ft]. Preferably this service road should be paved but may be surfaced with gravel. The road should be designed in such a way that the driver of an out-of-control vehicle will not mistake the service road for the arrester bed.

- Anchors, usually located adjacent to the arrester bed at 50- to 100-m [150- to 300-ft] intervals, are needed to secure a tow truck when removing a vehicle from the arrester bed. One anchor should be located about 30 m [100 ft] in advance of the bed to assist the wrecker in returning a captured vehicle to a surfaced roadway. The local tow-truck operators can be very helpful in properly locating the anchors.

As a vehicle rolls upgrade, it loses momentum and will eventually stop because of the effect of gravity. To determine the distance needed to bring the vehicle to a stop with consideration of the rolling resistance and gradient resistance, the following simplified equation may be used (*61*):

Metric	U.S. Customary	
$L = \dfrac{V^2}{254(R-G)}$	$L = \dfrac{V^2}{30(R-G)}$	(3-39)
where:	where:	
L = length of arrester bed, m	L = length of arrester bed, ft	
V = entering velocity, km/h	V = entering velocity, mph	
R = rolling resistance, expressed as equivalent percent gradient divided by 100 (see Table 3-33)	R = rolling resistance, expressed as equivalent percent gradient divided by 100 (see Table 3-33)	
G = percent grade divided by 100	G = percent grade divided by 100	

For example, assume that topographic conditions at a site selected for an emergency escape ramp limit the ramp to an upgrade of 10 percent ($G = +0.10$). The arrester bed is to be constructed with loose gravel for an entering speed of 140 km/h [90 mph]. Using Table 3-33, R is determined to be 0.10. The length of the arrester bed should be determined using Equation 3-39. For this example, the length of the arrester bed is about 400 m [1,350 ft].

When an arrester bed is constructed using more than one grade along its length, as shown in Figure 3-40, the speed loss occurring on each of the grades as the vehicle traverses the bed should be determined using the following equation:

Metric	U.S. Customary	
$V_f^2 = V_i^2 - 254L(R \pm G)$	$V_f^2 = V_i^2 - 30L(R \pm G)$	(3-40)
where:	where:	
V_f = speed at end of grade, km/h	V_f = speed at end of grade, mph	
V_i = entering speed at beginning of grade, km/h	V_i = entering speed at beginning of grade, mph	
L = length of grade, m	L = length of grade, ft	
R = rolling resistance, expressed as equivalent percent gradient divided by 100 (see Table 3-33)	R = rolling resistance, expressed as equivalent percent gradient divided by 100 (see Table 3-33)	
G = percent grade divided by 100	G = percent grade divided by 100	

The final speed for one section of the ramp is subtracted from the entering speed to determine a new entering speed for the next section of the ramp and the calculation repeated at each change in grade on the ramp until sufficient length is provided to reduce the speed of the out-of-control vehicle to zero.

Figure 3-40 shows a plan and profile of an emergency escape ramp with typical appurtenances.

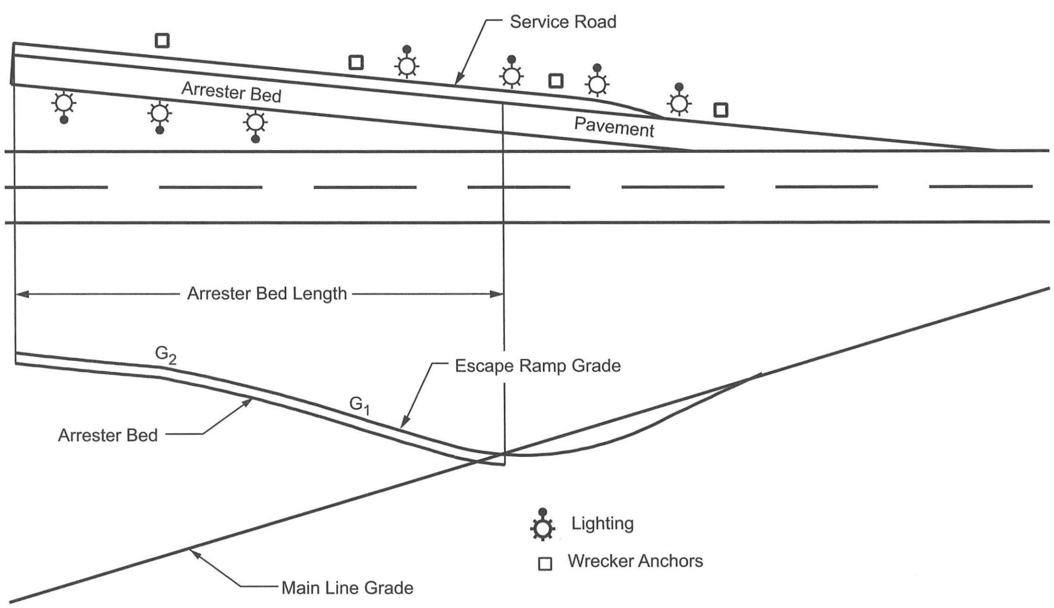


Figure 3-40. Typical Emergency Escape Ramp

Where the only practical location for an escape ramp will not provide sufficient length and grade to completely stop an out-of-control vehicle, it should be supplemented with an acceptable positive attenuation device.

Where a full-length ramp is to be provided with full deceleration capability for the design speed, a "last-chance" device should be considered when the consequences of leaving the end of the ramp are serious.

Any ramp-end treatment should be designed with care so that its advantages outweigh its disadvantages. The risk to others as the result of an out-of-control truck overrunning the end of an escape ramp may be more important than the harm to the driver or cargo of the truck. The abrupt deceleration of an out-of-control truck may cause shifting of the load, shearing of the fifth wheel, or jackknifing, all with potentially harmful occurrences to the driver and cargo.

Mounds of bedding material between 0.6 and 1.5 m [2 and 5 ft] high with 1V:1.5H slopes (i.e., slopes that change in elevation by one unit of length for each 1 to 1.5 units of horizontal distance) have been used at the end of ramps in several instances as the "last-chance" device. At least one escape ramp has been constructed with an array of crash cushions installed to prevent an out-of-control vehicle from leaving the end of the ramp. Furthermore, at the end of a hard-surfaced gravity ramp, a gravel bed or an attenuator array may sufficiently immobilize a brakeless runaway vehicle to keep it from rolling backward and jackknifing. Where barrels are used, the barrels should be filled with the same material as used in the arrester bed, so that any finer material does not result in contamination of the bed and reduction of the expected rolling resistance.

Brake-Check Areas

Turnouts or pulloff areas at the summit of a grade can be used for brake-check areas or mandatory-stop areas to provide an opportunity for a driver to inspect equipment on the vehicle and check that the brakes are not overheated at the beginning of the descent. In addition, information about the grade ahead and the location of escape ramps can be provided by diagrammatic signing or self-service pamphlets. An elaborate design is not needed for these areas. A brake-check area can be a paved lane behind and separated from the shoulder or a widened shoulder where a truck can stop. Appropriate signing should be used to discourage casual stopping by the public.

Maintenance

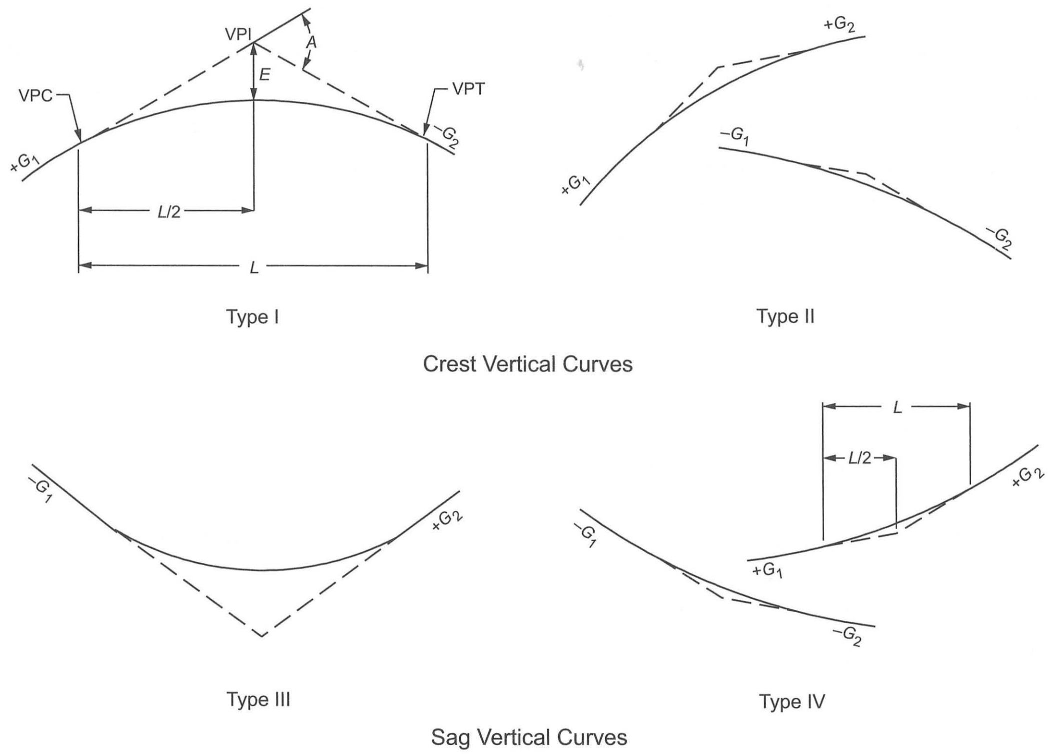
After each use, aggregate arrester beds should be reshaped using power equipment to the extent practical and the aggregate scarified as appropriate. Since aggregate tends to compact over time, the bedding material should be cleaned of contaminants and scarified periodically to retain the retarding characteristics of the bedding material and maintain free drainage. Using power equipment for work in the arrester bed reduces the exposure time for the maintenance workers to the potential that a runaway truck may need to use the facility. Maintenance of the appurtenances should be accomplished as appropriate.

3.4.6 Vertical Curves

General Considerations

Vertical curves to effect gradual changes between tangent grades may be any one of the crest or sag types depicted in Figure 3-41. Vertical curves should be simple in application and should result in a design that enables the driver to see the road ahead, enhances vehicle control, is pleasing in appearance, and is adequate for drainage. The major design control for crest vertical curves is the provision of ample sight distances for the design speed; while research (*17*) has shown that vertical curves with limited sight distance do not necessarily experience frequent crashes, it is recommended that all vertical curves should be designed to provide at least the stopping sight distances shown in Table 3-1. Wherever practical, longer stopping sight distances should be used. Furthermore, additional sight distance should be provided at decision points.

For driver comfort, the rate of change of grade should be kept within tolerable limits. This consideration is most important in sag vertical curves where gravitational and vertical centripetal forces act in opposite directions. Appearance also should be considered in designing vertical curves. A long curve has a more pleasing appearance than a short one; short vertical curves may give the appearance of a sudden break in the profile due to the effect of foreshortening.



G_1 and G_2 = Tangent Grades in Percent
A = Algebraic Difference in Grade
L = Length of Vertical Curve
E = Vertical Offset at the VPI

Figure 3-41. Types of Vertical Curves

Drainage of curbed roadways on sag vertical curves (Type III in Figure 3-41) needs careful profile design to retain a grade of not less than 0.5 percent or, in some cases, 0.30 percent for the outer edges of the roadway. Although not desirable, flatter grades may be appropriate in some situations.

For simplicity, a parabolic curve with an equivalent vertical axis centered on the Vertical Point of Intersection (VPI) is usually used in roadway profile design. The vertical offsets from the tangent vary as the square of the horizontal distance from the curve end (Point of Tangency). The vertical offset from the tangent grade at any point along the curve is calculated as a proportion of the vertical offset at the VPI, which is $AL/800$, where the symbols are as shown in Figure 3-41. The rate of change of grade at successive points on the curve is a constant amount for equal increments of horizontal distance, and is equal to the algebraic difference between intersecting tangent grades divided by the length of curve in meters [feet], or A/L in percent per meter [percent per foot]. The reciprocal L/A is the horizontal distance in meters [feet] needed to make a 1 percent change in gradient and is, therefore, a measure of curvature. The quantity L/A, termed "K," is useful in determining the horizontal distance from the Vertical Point of Curvature (VPC) to the high point of Type I curves or to the low point of Type III curves. This point where the slope is zero occurs at a distance from the VPC equal to K times the approach gradient. The value of K is also useful in determining minimum lengths of vertical curves for various design speeds. Other details on parabolic vertical curves are found in textbooks on highway engineering.

In certain situations, because of critical clearance or other controls, the use of asymmetrical vertical curves may be appropriate. Because the conditions under which such curves are appropriate are infrequent, the derivation and use of the relevant equations have not been included herein. For use in such limited instances, refer to asymmetrical curve data found in a number of highway engineering texts.

Crest Vertical Curves

Minimum lengths of crest vertical curves based on sight distance criteria generally are satisfactory from the standpoint of safety, comfort, and appearance. An exception may be at decision areas, such as ramp exit gores, where longer sight distances and, therefore, longer vertical curves should be provided; for further information, refer to Section 3.2.3, "Decision Sight Distance."

Figure 3-42 illustrates the parameters used in determining the length of a parabolic crest vertical curve needed to provide any specified value of sight distance. The basic equations for length of a crest vertical curve in terms of algebraic difference in grade and sight distance follow:

Metric	U.S. Customary	
When S is less than L,	When S is less than L,	
$L=\dfrac{AS^2}{100\left(\sqrt{2h_1}+\sqrt{2h_2}\right)^2}$	$L=\dfrac{AS^2}{100\left(\sqrt{2h_1}+\sqrt{2h_2}\right)^2}$	(3-41)
When S is greater than L,	When S is greater than L,	
$L=2S-\dfrac{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2}{A}$	$L=2S-\dfrac{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2}{A}$	(3-42)
where:	where:	
L = length of vertical curve, m	L = length of vertical curve, ft	
A = algebraic difference in grades, percent	A = algebraic difference in grades, percent	
S = sight distance, m	S = sight distance, ft	
h_1 = height of eye above roadway surface, m	h_1 = height of eye above roadway surface, ft	
h_2 = height of object above roadway surface, m	h_2 = height of object above roadway surface, ft	

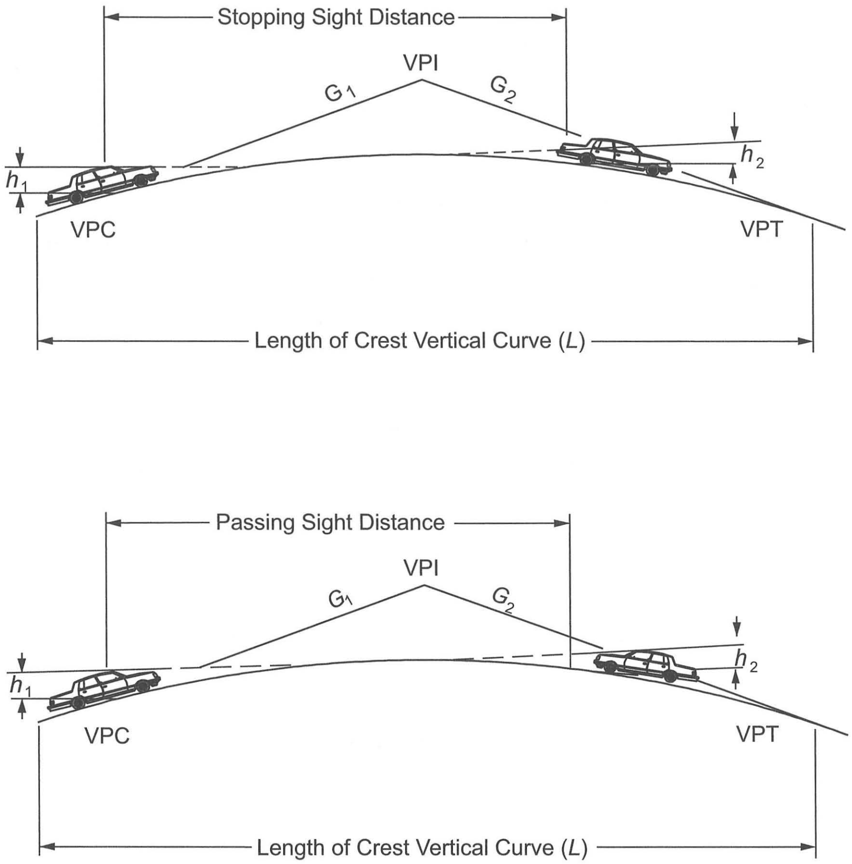


Figure 3-42. Parameters Considered in Determining the Length of a Crest Vertical Curve to Provide Sight Distance

When the height of eye and the height of object are 1.08 and 0.60 m [3.50 ft and 2.00 ft], respectively, as used for stopping sight distance, the equations become:

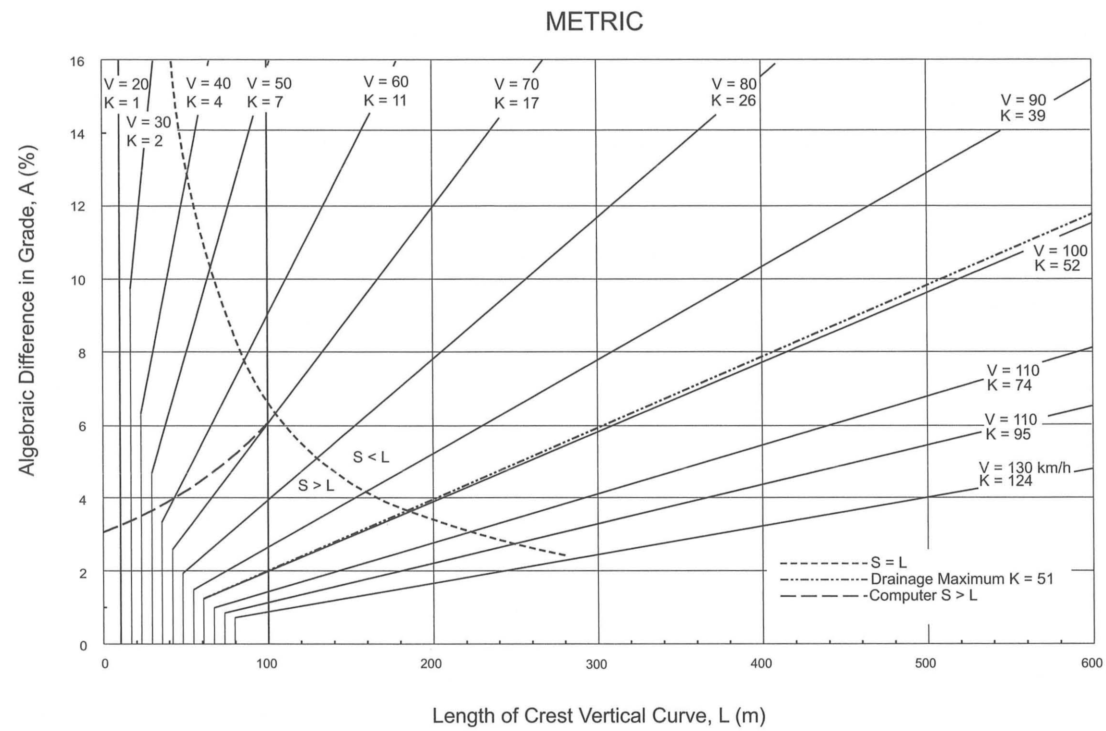
Metric	U.S. Customary	
When S is less than L,	When S is less than L,	
$L = \frac{AS^2}{658}$	$L = \frac{AS^2}{2158}$	(3-43)
When S is greater than L,	When S is greater than L,	
$L = 2S - \frac{658}{A}$	$L = 2S - \frac{2158}{A}$	(3-44)

Design controls: stopping sight distance—The minimum lengths of crest vertical curves for different values of A to provide the minimum stopping sight distances for each design speed are shown in Figure 3-43. The solid lines give the minimum vertical curve lengths, on the basis of rounded values of K as determined from Equations 3-43 and 3-44.

The short dashed curve at the lower left, crossing these lines, indicates where $S = L$. Note that to the right of the $S = L$ line, the value of K, or length of vertical curve per percent change in A, is a simple and convenient expression of the design control. For each design speed this single value is a positive whole number that is indicative of the rate of vertical curvature. The design control in terms of K covers all combinations of A and L for any one design speed; thus, A and L need not be indicated separately in a tabulation of design value. The selection of design curves is facilitated because the minimum length of curve in meters [feet] is equal to K times the algebraic difference in grades in percent, $L = KA$. Conversely, the checking of plans is simplified by comparing all curves with the design value for K.

Table 3-34 shows the computed K values for lengths of vertical curves corresponding to the stopping sight distances shown in Table 3-1 for each design speed. For direct use in design, values of K are rounded as shown in the right column. The rounded values of K are plotted as the solid lines in Figure 3-43. These rounded values of K are higher than computed values, but the differences are not significant.

Where S is greater than L (lower left in Figure 3-43), the computed values plot as a curve (as shown by the dashed line for 70 km/h [45 mph]) that bends to the left, and for small values of A, the vertical curve lengths are zero because the sight line passes over the high point. This relationship does not represent desirable design practice. Most states use a minimum length of vertical curve, expressed as a single value, a range for different design speeds, or a function of A. Values now in use range from about 30 to 100 m [100 to 325 ft]. To recognize the distinction in design speed and to approximate the range of current practice, minimum lengths of vertical curves are expressed as about 0.6 times the design speed in km/h, $L_{min} = 0.6V$, where V is in kilometers per hour and L is in meters, or about three times the design speed in mph, [$L_{min} = 3V$], where V is in miles per hour and L is in feet. These terminal adjustments show as the vertical lines at the lower left of Figure 3-43.



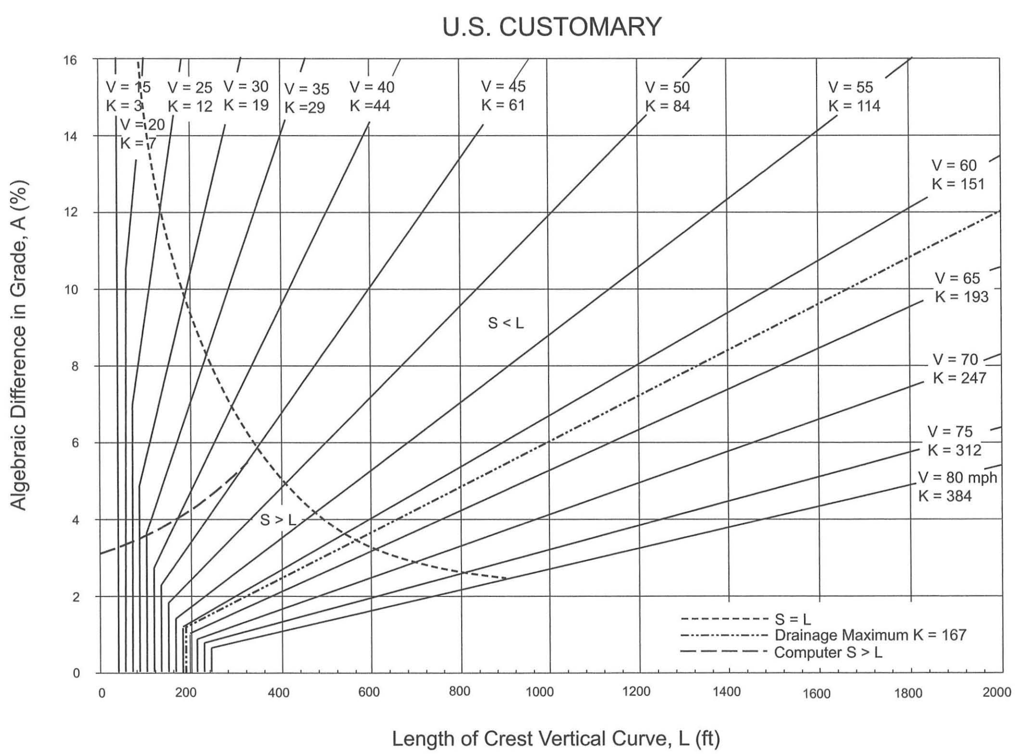


Figure 3-43. Design Controls for Crest Vertical Curves—Open Road Conditions

Table 3-34. Design Controls for Crest Vertical Curves Based on Stopping Sight Distance

Metric				U.S. Customary			
Design Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature, K^a		Design Speed (mph)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, K^a	
		Calculated	Design			Calculated	Design
20	20	0.6	1	15	80	3.0	3
30	35	1.9	2	20	115	6.1	7
40	50	3.8	4	25	155	11.1	12
50	65	6.4	7	30	200	18.5	19
60	85	11.0	11	35	250	29.0	29
70	105	16.8	17	40	305	43.1	44
80	130	25.7	26	45	360	60.1	61
90	160	38.9	39	50	425	83.7	84
100	185	52.0	52	55	495	113.5	114
110	220	73.6	74	60	570	150.6	151
120	250	95.0	95	65	645	192.8	193
130	285	123.4	124	70	730	246.9	247
				75	820	311.6	312
				80	910	383.7	384

[a] Rate of vertical curvature, K, is the length of curve per percent algebraic difference in intersecting grades (A), $K = L/A$.

The values of K derived above when S is less than L also can be used without significant error where S is greater than L. As shown in Figure 3-42, extension of the diagonal lines to meet the vertical lines for minimum lengths of vertical curves results in appreciable differences from the theoretical only where A is small and little or no additional cost is involved in obtaining longer vertical curves.

For night driving on highways without lighting, the length of visible roadway is that roadway that is directly illuminated by the headlights of the vehicle. For certain conditions, the minimum stopping sight distance values used for design exceed the length of visible roadway. First, vehicle headlights have limitations on the distance over which they can project the light intensity levels that are needed for visibility. When headlights are operated on low beams, the reduced candlepower at the source plus the downward projection angle significantly restrict the length of visible roadway surface. Thus, particularly for high-speed conditions, stopping sight distance values exceed road-surface visibility distances afforded by the low-beam headlights regardless of whether the roadway profile is level or curving vertically. Second, for crest vertical curves, the area forward of the headlight beam's point of tangency with the roadway surface is shadowed and receives only indirect illumination.

Since the headlight mounting height (typically about 0.60 m [2.00 ft]) is lower than the driver eye height used for design (1.08 m [3.50 ft]), the sight distance to an illuminated object is controlled by the height of the vehicle headlights rather than by the direct line of sight. Any object within the shadow zone must be high enough to extend into the headlight beam to be directly illuminated. On the basis of Equation 3-41, the bottom of the headlight beam is about 0.40 m [1.30 ft] above the roadway at a distance ahead of the vehicle equal to the stopping sight distance. Although the vehicle headlight system does limit roadway visibility length as previously mentioned, there is some mitigating effect in that other vehicles, whose taillight height typically varies from 0.45 to 0.60 m [1.50 to 2.00 ft], and other sizable objects receive direct lighting from headlights at stopping sight distance values used for design. Furthermore, drivers are aware that visibility at night is less than during the day, regardless of road and street design features, and they may therefore be more attentive and alert.

There is a level point on a crest vertical curve of Type I (see Figure 3-41), but no difficulty with drainage on highways with curbs is typically experienced if the curve is sharp enough so that a minimum grade of 0.30 percent is reached at a point about 15 m [50 ft] from the crest. This corresponds to K of 51 m [167 ft] per percent change in grade, which is plotted in Figure 3-43 as the drainage maximum. All combinations above or to the left of this line satisfy the drainage criterion. The combinations below and to the right of this line involve flatter vertical curves. Special attention is needed in these cases to provide proper pavement drainage near the high point of crest vertical curves. It is not intended that K of 51 m [167 ft] per percent grade be considered a design maximum, but merely a value beyond which drainage should be more carefully designed.

Design controls: passing sight distance—Design values of crest vertical curves for passing sight distance differ from those for stopping sight distance because of the different sight distance and object height criteria. The general Equations 3-41 and 3-42 apply. Using the 1.08-m [3.50-ft] height of object results in the following specific formulas with the same terms as shown above:

Metric	U.S. Customary	
When S is less than L,	When S is less than L,	
$L=\dfrac{AS^2}{864}$	$L=\dfrac{AS^2}{2800}$	(3-45)
When S is greater than L,	When S is greater than L,	
$L=2S-\dfrac{864}{A}$	$L=2S-\dfrac{2800}{A}$	(3-46)

For the minimum passing sight distances shown in Table 3-4, the minimum lengths of crest vertical curves are substantially longer than those for stopping sight distances. The extent of difference is evident by the values of K, or length of vertical curve per percent change in A, for passing sight distances shown in Table 3-35.

Table 3-35. Design Controls for Crest Vertical Curves Based on Passing Sight Distance

Metric			U.S. Customary		
Design Speed (km/h)	Passing Sight Distance (m)	Rate of Vertical Curvature, K^a Design	Design Speed (mph)	Passing Sight Distance (ft)	Rate of Vertical Curvature, K^a Design
30	120	17	20	400	57
40	140	23	25	450	72
50	160	30	30	500	89
60	180	38	35	550	108
70	210	51	40	600	129
80	245	69	45	700	175
90	280	91	50	800	229
100	320	119	55	900	289
110	355	146	60	1000	357
120	395	181	65	1100	432
130	440	224	70	1200	514
			75	1300	604
			80	1400	700

[a] Rate of vertical curvature, K, is the length of curve per percent algebraic difference in intersecting grades (A), $K = L/A$.

Generally, it is impractical to design crest vertical curves that provide passing sight distance because of high cost where crest cuts are involved and the difficulty of fitting the resulting long vertical curves to the terrain, particularly for high-speed roads. Passing sight distance on crest vertical curves may be practical on roads with unusual combinations of low design speeds and gentle grades or higher design speeds with very small algebraic differences in grades. Ordinarily, passing sight distance is provided only at locations where combinations of alignment and profile do not need significant grading. Table 3-35 shows computed K values for determining lengths of vertical curves corresponding to passing sight distance values shown in Table 3-4.

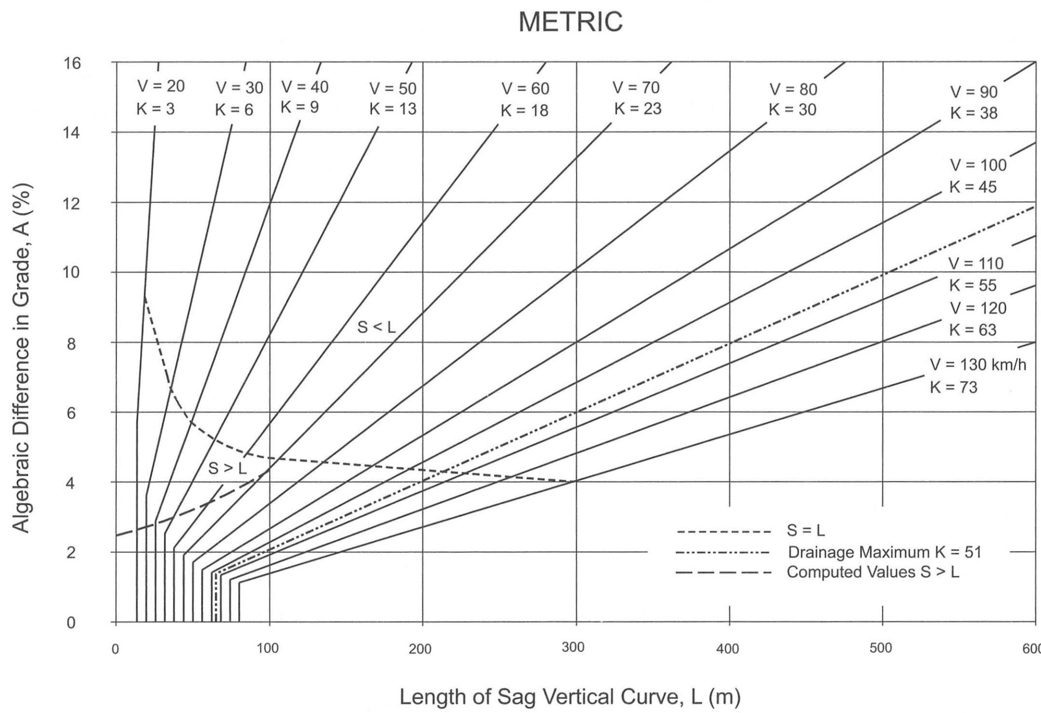
Sag Vertical Curves

At least four different criteria for establishing lengths of sag vertical curves are recognized to some extent. These are (1) headlight sight distance, (2) passenger comfort, (3) drainage control, and (4) general appearance.

Headlight sight distance has been used directly by some agencies and for the most part is the basis for determining the length of sag vertical curves recommended here. When a vehicle traverses a sag vertical curve at night, the portion of highway lighted ahead is dependent on the position of the headlights and the direction of the light beam. A headlight height of 0.60 m [2 ft] and a 1-degree upward divergence of the light beam from the longitudinal axis of the vehicle is commonly assumed. The upward spread of the light beam above the 1-degree divergence angle provides some additional visible length of roadway, but is not generally considered in design. The following equations show the relationships between S, L, and A, using S as the distance between the vehicle and point where the 1-degree upward angle of the light beam intersects the surface of the roadway:

Metric	U.S. Customary	
When S is less than L, $$L = \frac{AS^2}{200\left[0.6 + S\left(\tan 1^\circ\right)\right]}$$	When S is less than L, $$L = \frac{AS^2}{200\left[2.0 + S\left(\tan 1^\circ\right)\right]}$$	(3-47)
or, $$L = \frac{AS^2}{120 + 3.5S}$$	or, $$L = \frac{AS^2}{400 + 3.5S}$$	(3-48)
When S is greater than L, $$L = 2S - \frac{200\left[0.6 + S\left(\tan 1^\circ\right)\right]}{A}$$	When S is greater than L, $$L = 2S - \frac{200\left[2.0 + S\left(\tan 1^\circ\right)\right]}{A}$$	(3-49)
or, $$L = 2S - \frac{120 + 3.5S}{A}$$	or, $$L = 2S - \frac{400 + 3.5S}{A}$$	(3-50)
where: L = length of sag vertical curve, m A = algebraic difference in grades, percent S = light beam distance, m	where: L = length of sag vertical curve, ft A = algebraic difference in grades, percent S = light beam distance, ft	

For drivers to see the roadway ahead, a sag vertical curve should be long enough that the light beam distance is approximately the same as the stopping sight distance. Accordingly, it is appropriate to use stopping sight distances for different design speeds as the value of S in the above equations. The resulting lengths of sag vertical curves for the recommended stopping sight distances for each design speed are shown in Figure 3-44 with solid lines using rounded values of K as was done for crest vertical curves.



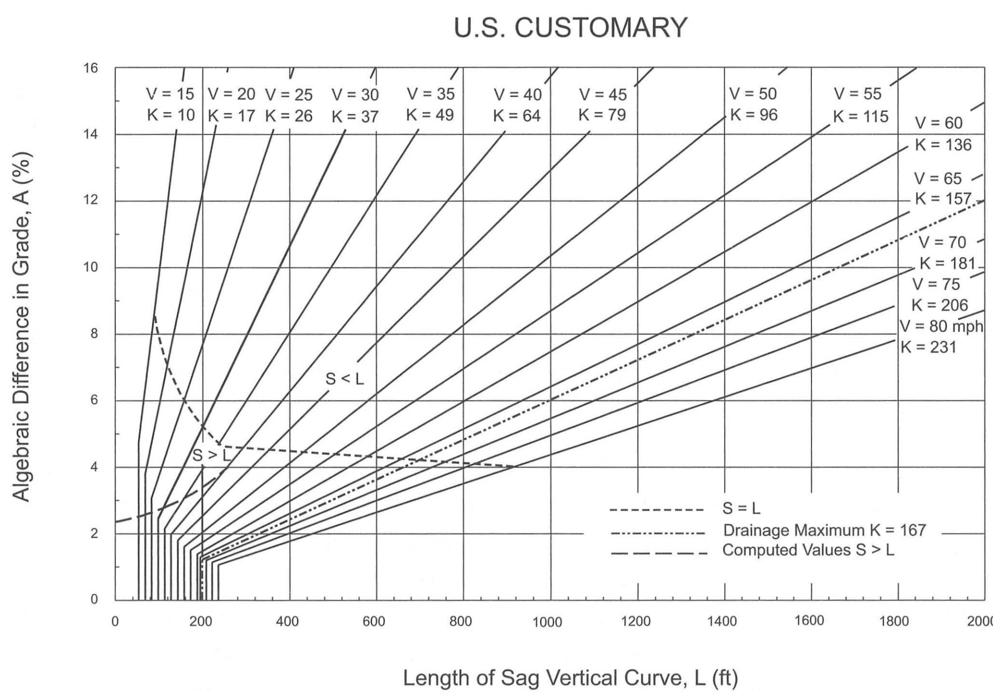


Figure 3-44. Design Controls for Sag Vertical Curves—Open Road Conditions

The effect on passenger comfort of the change in vertical direction is greater on sag than on crest vertical curves because gravitational and centripetal forces are combining rather than opposing forces. Comfort due to change in vertical direction is not easily measured because it is affected appreciably by vehicle body suspension, vehicle body weight, tire flexibility, and other factors. Limited attempts at such measurements have led to the broad conclusion that riding is comfortable on sag vertical curves when the centripetal acceleration does not exceed 0.3 m/s^2 [1 ft/s^2]. The general expression for such a criterion is:

Metric	U.S. Customary	
$L=\frac{AV^2}{395}$	$L=\frac{AV^2}{46.5}$	(3-51)
where:	where:	
L = length of sag vertical curve, m	L = length of sag vertical curve, ft	
A = algebraic difference in grades, percent	A = algebraic difference in grades, percent	
V = design speed, km/h	V = design speed, mph	

The length of vertical curve needed to satisfy this comfort factor at the various design speeds is only about 50 percent of that needed to satisfy the headlight sight distance criterion for the normal range of design conditions.

Drainage affects design of vertical curves of Type III (see Figure 3-42) where curbed sections are used. An approximate criterion for sag vertical curves is the same as that expressed for the crest conditions (i.e., a minimum grade of 0.30 percent should be provided within 15 m [50 ft] of the level point). This criterion corresponds to K of 51 m [167 ft] per percent change in grade, which is plotted in Figure 3-44 as the drainage maximum. The drainage criterion differs from other criteria in that the length of sag vertical curve determined for it is a maximum, whereas, the length for any other criterion is a minimum. The maximum length of the drainage criterion is greater than the minimum length for other criteria up to 100 km/h [65 mph].

For improved appearance of sag vertical curves, previous guidance used a rule-of-thumb for minimum curve length of $30A$ [$100A$] or, in Figure 3-44, $K = 30$ m [$K = 100$ ft] per percent change in grade. This approximation is a generalized control for small or intermediate values of A. Compared with headlight sight distance, it corresponds to a design speed of approximately 80 km/h [50 mph]. On high-type highways, longer curves are appropriate to improve appearance.

From the preceding discussion, it is evident that design controls for sag vertical curves differ from those for crests, and separate design values are needed. The headlight sight distance appears to be the most logical criterion for general use, and the values determined for stopping sight distances are within the limits recognized in current practice. The use of this criterion to establish design values for a range of lengths of sag vertical curves is recommended. As in the case of crest vertical curves, it is convenient to express the design control in terms of the K rate for all values of A. This entails some deviation from the computed values of K for small values of A, but the differences are not significant. Table 3-36 shows the range of computed values and the rounded values of K selected as design controls. The lengths of sag vertical curves on the basis of the design speed values of K are shown by the solid lines in Figure 3-44. It is to be emphasized that these lengths are minimum values based on design speed; longer curves are desired wherever practical, but special attention to drainage should be exercised where values of K in excess of 51 m [167 ft] per percent change in grade are used.

Minimum lengths of vertical curves for flat gradients also are recognized for sag conditions. The values determined for crest conditions appear to be generally suitable for sags. Lengths of sag vertical curves, shown as vertical lines in Figure 3-44, are equal to 0.6 times the design speed in km/h [three times the design speed in mph].

Sag vertical curves shorter than the lengths computed from Table 3-36 may be justified for economic reasons in cases where an existing feature, such as a structure not ready for replacement, controls the vertical profile. In certain cases, ramps may also be designed with shorter sag vertical curves. Fixed-source lighting is desirable in such cases. For street design, some engineers accept design of a sag or crest where A is about 1 percent or less without a length of calculated vertical curve. However, field modifications during construction usually result in constructing the equivalent to a vertical curve, even if short.

Table 3-36. Design Controls for Sag Vertical Curves

Metric				U.S. Customary			
Design Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature, K^a		Design Speed (mph)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, K^a	
		Calculated	Design			Calculated	Design
20	20	2.1	3	15	80	9.4	10
30	35	5.1	6	20	115	16.5	17
40	50	8.5	9	25	155	25.5	26
50	65	12.2	13	30	200	36.4	37
60	85	17.3	18	35	250	49.0	49
70	105	22.6	23	40	305	63.4	64
80	130	29.4	30	45	360	78.1	79
90	160	37.6	38	50	425	95.7	96
100	185	44.6	45	55	495	114.9	115
110	220	54.4	55	60	570	135.7	136
120	250	62.8	63	65	645	156.5	157
130	285	72.7	73	70	730	180.3	181
				75	820	205.6	206
				80	910	231.0	231

[a] Rate of vertical curvature, K, is the length of curve (m) per percent algebraic difference intersecting grades (A), $K = L/A$.

Sight Distance at Undercrossings

Sight distance on the highway through a grade separation should be at least as long as the minimum stopping sight distance and preferably longer. Design of the vertical alignment is the same as at any other point on the highway except in some cases of sag vertical curves underpassing a structure as illustrated in Figure 3-45. While not a frequent concern, the structure fascia may cut the line of sight and limit the sight distance to less than otherwise is attainable. It is generally practical to provide the minimum length of sag vertical curve at grade separation structures, and even where the recommended grades are exceeded, the sight distance should not need to be reduced below the minimum recommended values for stopping sight distance.

For some conditions, the designer may wish to check the available sight distance at an undercrossing, such as at a two-lane undercrossing without ramps where it would be desirable to provide passing sight distance. Such checks are best made graphically on the profile, but may be performed through computations.

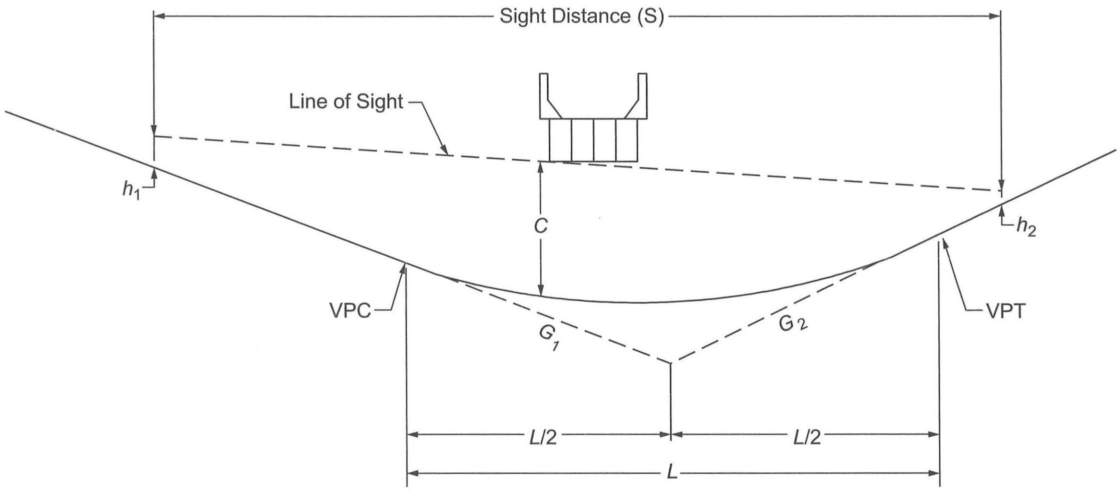


Figure 3-45. Sight Distance at Undercrossings

The general equations for sag vertical curve length at undercrossings are:

Case 1—Sight distance greater than length of vertical curve ($S > L$):

Metric	U.S. Customary	
$$L=2S-\frac{800\left[C-\left(\frac{h_1+h_2}{2}\right)\right]}{A}$$	$$L=2S-\frac{800\left[C-\left(\frac{h_1+h_2}{2}\right)\right]}{A}$$	(3-52)
where: L = length of vertical curve, m S = sight distance, m C = vertical clearance, m h_1 = height of eye, m h_2 = height of object, m A = algebraic difference in grades, percent	where: L = length of vertical curve, ft S = sight distance, ft C = vertical clearance, ft h_1 = height of eye, ft h_2 = height of object, ft A = algebraic difference in grades, percent	

Case 2—Sight distance less than length of vertical curve ($S < L$):

Metric	U.S. Customary	
$L = \dfrac{AS^2}{800\left[C - \left(\dfrac{h_1 + h_2}{2}\right)\right]}$	$L = \dfrac{AS^2}{800\left[C - \left(\dfrac{h_1 + h_2}{2}\right)\right]}$	(3-53)
where:	where:	
L = length of vertical curve, m	L = length of vertical curve, ft	
A = algebraic difference in grades, percent	A = algebraic difference in grades, percent	
S = sight distance, m	S = sight distance, ft	
C = vertical clearance, m	C = vertical clearance, ft	
h_1 = height of eye, m	h_1 = height of eye, ft	
h_2 = height of object, m	h_2 = height of object, ft	

Using an eye height of 2.4 m [8.0 ft] for a truck driver and an object height of 0.6 m [2.0 ft] for the taillights of a vehicle, the following equations can be derived:

Case 1—Sight distance greater than length of vertical curve ($S > L$):

Metric	U.S. Customary	
$L = 2S - \dfrac{800(C - 1.5)}{A}$	$L = 2S - \dfrac{800(C - 5)}{A}$	(3-54)

Case 2—Sight distance less than length of vertical curve ($S < L$):

Metric	U.S. Customary	
$L = \dfrac{AS^2}{800(C - 1.5)}$	$L = \dfrac{AS^2}{800(C - 5)}$	(3-55)

General Controls for Vertical Alignment

In addition to the specific controls for vertical alignment discussed previously, there are several general controls that should be considered in design.

- A smooth gradeline with gradual changes, as consistent with the type of highway, road, or street and the character of terrain, should be sought for in preference to a line with numerous breaks and short lengths of grades. Specific design criteria are the maximum grade and the critical length of grade, but the manner in which they are applied and fitted to the terrain on a continuous line determines the suitability and appearance of the finished product.

- The "roller-coaster" or the "hidden-dip" type of profile should be avoided. Such profiles generally occur on relatively straight, horizontal alignment where the roadway profile closely follows a rolling natural ground line. Examples of such undesirable profiles are evident on many older roads and streets; they are unpleasant aesthetically and difficult to drive. Hidden dips may create difficulties for drivers who wish to pass, because the passing driver may be deceived if the view of the road or street beyond the dip is free of opposing vehicles. Even with shallow dips, this type of profile may be disconcerting, because the driver cannot be sure whether or not there is an oncoming vehicle hidden beyond the rise. This type of profile is avoided by use of horizontal curves or by more gradual grades.

- Undulating gradelines, involving substantial lengths of momentum grades, should be evaluated for their effect on traffic operation. Such profiles permit heavy trucks to operate at higher overall speeds than where an upgrade is not preceded by a downgrade, but may encourage excessive speeds of trucks with attendant conflicts with other traffic.

- A "broken-back" gradeline (two vertical curves in the same direction separated by a short section of tangent grade) generally should be avoided, particularly in sags where the full view of both vertical curves is not pleasing. This effect is particularly noticeable on divided roadways with open median sections.

- On long grades, it may be preferable to place the steepest grades at the bottom and flatten the grades near the top of the ascent or to break the sustained grade by short intervals of flatter grade instead of providing a uniform sustained grade that is only slightly below the recommended maximum. This is particularly applicable to roads and streets with low design speeds.

- Where at-grade intersections occur on roadway sections with moderate to steep grades, it is desirable to reduce the grade through the intersection. Such profile changes are beneficial for vehicles making turns and serve to reduce the potential for crashes.

- Sag vertical curves should be avoided in cuts unless adequate drainage can be provided.

3.5 COMBINATIONS OF HORIZONTAL AND VERTICAL ALIGNMENT

3.5.1 General Considerations

Horizontal and vertical alignment are permanent design elements for which thorough study is warranted. It is extremely difficult and costly to correct alignment deficiencies after a highway is constructed. On freeways, there are numerous controls such as multilevel structures and costly right-of-way. On most arterial streets, heavy development takes place along the property lines, which makes it impractical to change the alignment in the future. Thus, compromises in the alignment designs should be weighed carefully because any initial savings may be more than offset by the economic loss to the public in the form of crashes and delays.

Horizontal and vertical alignment should not be designed independently. They complement each other, and poorly designed combinations can spoil the good points and aggravate the deficiencies of each. Horizontal alignment and profile are among the more important of the permanent design elements of the highway. Excellence in the design of each and of their combination enhances vehicle control, encourages uniform speed, and improves appearance, nearly always without additional cost (*1*, *10*, *15*, *41*, *54*, *55*, *63*, *64*).

3.5.2 General Design Controls

It is difficult to discuss combinations of horizontal alignment and profile without reference to the broader issue of highway location. These subjects are interrelated and what is said about one is generally applicable to the other. It is assumed in this discussion that the general location of a facility has been fixed and that the remaining task is the development of a specific design harmonizing of the vertical and horizontal lines such that the finished highway, road, or street will be an economical, pleasant, and safe facility on which to travel. The physical constraints or influences that act singly or in combination to determine the alignment are: the character of roadway based on the traffic, topography, and subsurface conditions; the existing cultural development; likely future developments; and the location of the roadway's terminals. Design speed is considered in determining the general roadway location, but as design proceeds to the development of more detailed alignment and profile it assumes greater importance. The selected design speed serves to keep all elements of design in balance. Design speed determines limiting values for many elements such as curvature and sight distance and influences many other elements such as width, clearance, and maximum gradient, which are all discussed in the preceding portions of this chapter.

Appropriate combinations of horizontal alignment and profile are obtained through engineering studies and consideration of the following general guidelines:

- Curvature and grades should be in proper balance. Tangent alignment or flat curvature at the expense of steep or long grades and excessive curvature with flat grades both represent poor design. A logical design that offers the best combination of safety, capacity, ease and uniformity of operation, and pleasing appearance within the practical limits of terrain and area traversed is a compromise between these two extremes.

- Vertical curvature superimposed on horizontal curvature, or vice versa, generally results in a more pleasing facility, but such combinations should be analyzed for their effect on traffic. Successive changes in profile not in combination with horizontal curvature may result in a series of humps visible to the driver for some distance which represents an undesirable condition.

- Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. This condition is undesirable because the driver may not perceive the horizontal change in alignment, especially at night. The disadvantages of this arrangement are avoided if the horizontal curvature leads the vertical curvature (i.e., the horizontal curve is made longer than the vertical curve). Suitable designs can also be developed by using design values well above the appropriate minimum values for the design speed.

- Somewhat related to the preceding guideline, sharp horizontal curvature should not be introduced near the bottom of a steep grade approaching or near the low point of a pronounced sag vertical curve. Because the view of the road ahead is foreshortened, any horizontal curvature other than a very flat curve assumes an undesirable distorted appearance. Further, vehicle speeds, particularly for trucks, are often high at the bottom of grades, and erratic operations may result, especially at night.

- On two-lane roads and streets, the need for passing sections at frequent intervals and including an appreciable percentage of the length of the roadway often supersedes the general guidelines for combinations of horizontal and vertical alignment. In such cases, it is appropriate to work toward long tangent sections to assure sufficient passing sight distance in design.
- Both horizontal curvature and profile should be made as flat as practical at intersections where sight distance along either roads or streets is important and vehicles may have to slow or stop.
- On divided highways and streets, variation in width of median and the use of independent profiles and horizontal alignments for the separate one-way roadways are sometimes desirable. Where traffic justifies provision of four lanes, a superior design without additional cost generally results from such practices.
- In residential areas, the alignment should be designed to minimize nuisance to the neighborhood. Generally, a depressed facility makes a highway less visible and less noisy to adjacent residents. Minor horizontal adjustments can sometimes be made to increase the buffer zone between the highway and clusters of homes.
- The alignment should be designed to enhance attractive scenic views of the natural and manmade environment, such as rivers, rock formations, parks, and outstanding structures. The highway should head into, rather than away from, those views that are outstanding; it should fall toward those features of interest at a low elevation, and it should rise toward those features best seen from below or in silhouette against the sky.

3.5.3 Alignment Coordination in Design

Coordination of horizontal alignment and profile should not be left to chance but should begin with preliminary design, at which time adjustments can be readily made. Although a specific 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 PC 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 operation, 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 PC 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 PC 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. The crossroad or street intersections and the location of driveways are dominant 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 3-46.

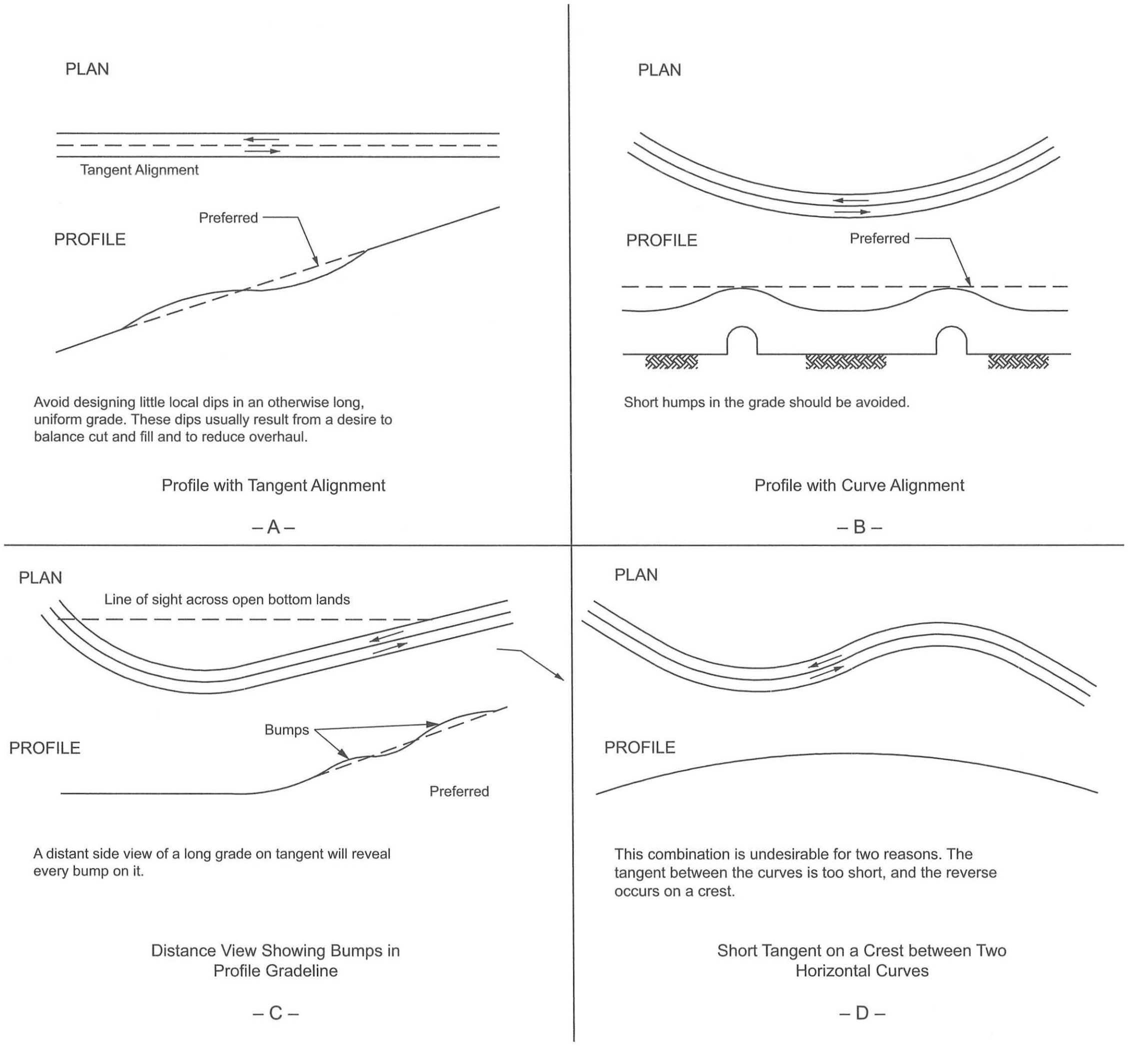


Figure 3-46. Alignment and Profile Relationships in Roadway Design (*41*)

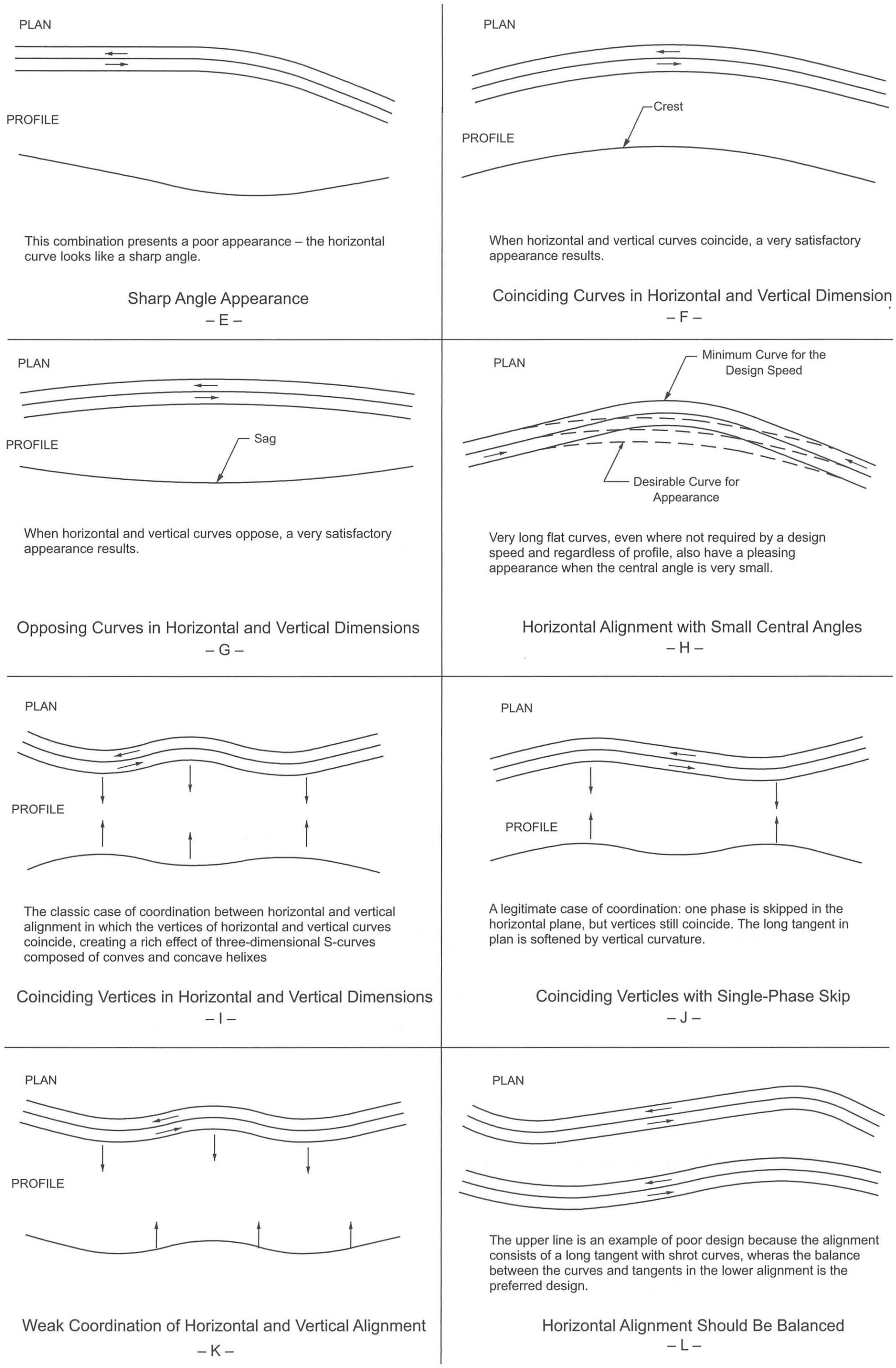
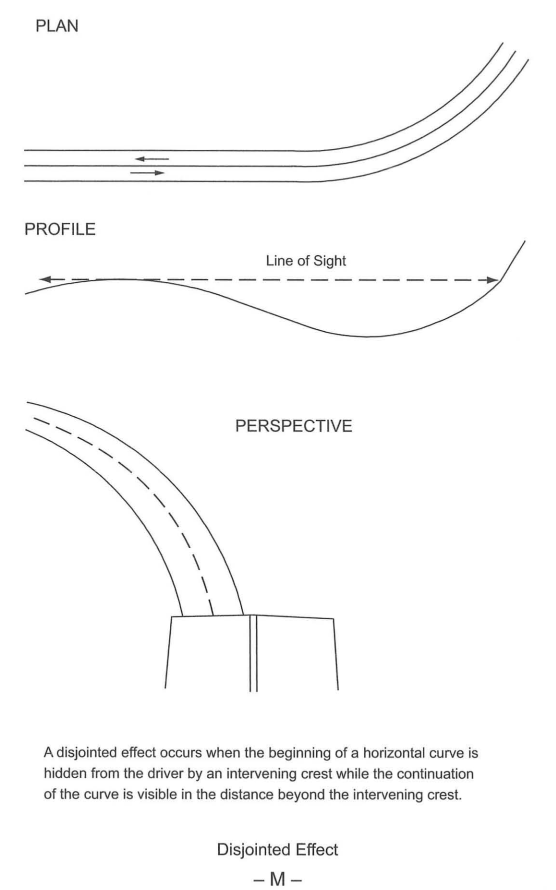


Figure 3-46. Alignment and Profile Relationships in Roadway Design (Continued)



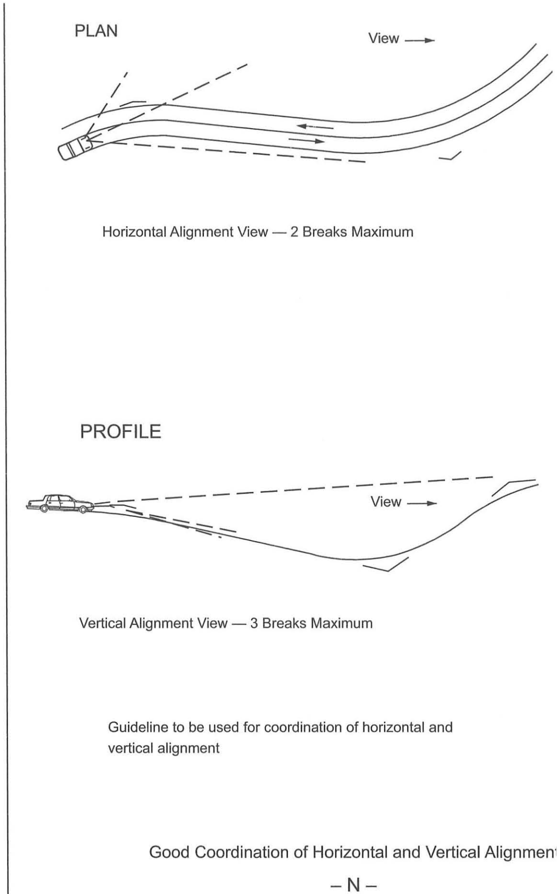


Figure 3-46. Alignment and Profile Relationships in Roadway Design (Continued)

3.6 OTHER FEATURES AFFECTING GEOMETRIC DESIGN

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

3.6.1 Erosion Control and Landscape Development

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

Erosion and maintenance are minimized largely by using specific design features: flat side slopes, rounded and blended with natural terrain; serrated cut slopes; drainage channels designed with due regard to width, depth, slopes, alignment, and protective treatment; inlets located and spaced with erosion control in mind; prevention of erosion at culvert outlets; proper facilities for groundwater interception; dikes, berms, and other protective devices to trap sediment at strategic locations; and protective ground covers and planting. To the extent practical, these features should be designed and located to minimize the potential crash severity for motorists who unintentionally run off the roadway.

Landscape development should be in keeping with the character of the highway and its environment. Programs include the following general areas of improvement: (1) preservation of existing vegetation, (2) transplanting of existing vegetation where practical, (3) planting of new vegetation, (4) selective clearing and thinning, and (5) regeneration of natural plant species and material.

The objectives in planting or the retention and preservation of natural growth on roadsides are closely related. In essence, they provide vegetation that (1) will be an aid to aesthetics; (2) will aid in lowering construction and maintenance costs; and (3) create interest, usefulness, and beauty for the pleasure and satisfaction of the traveling public without increasing the potential crash severity for motorists who unintentionally run off the roadway.

Landscaping of urban highways and streets assumes additional importance in mitigating the many nuisances associated with urban traffic. Landscaping can reduce this contribution to urban blight and make the urban highways and streets better neighbors.

Further information concerning landscape development and erosion control is presented in the AASHTO *Guide for Transportation Landscape and Environmental Design* (*1*).

3.6.2 Rest Areas, Information Centers, and Scenic Overlooks

Rest areas, information centers, and scenic overlooks are functional and desirable elements of the complete highway facility and are provided to reduce driver fatigue and for the convenience of highway users. A safety rest area is a roadside area, with parking facilities separated from the roadway, provided for the travelers to stop and rest for short periods. The area may provide drinking water, restrooms, tables and benches, telephones, information displays, and other facilities for travelers. A rest area is not intended to be used for social or civic gatherings or for such active forms of recreation as boating, swimming, or organized games. An information center is a staffed or unstaffed facility at a rest area for the purpose of furnishing travel and other information or services to travelers. A scenic overlook is a roadside area provided for motorists to park their vehicles, beyond the shoulder, primarily for viewing the scenery or for taking photographs in a location removed from through traffic. Scenic overlooks need not provide comfort and convenience facilities.

Site selection for rest areas, information centers, and scenic overlooks should consider the scenic quality of the area, accessibility, and adaptability to development. Other essential considerations include an adequate source of water and a means to treat and/or properly dispose of sewage. Site plans should be developed through the use of a comprehensive site planning process that should include the location of ramps, parking areas for cars and trucks, buildings, picnic areas, water supply, sewage treatment facilities, and maintenance areas. The objective is to give maximum weight to the appropriateness of the site rather than adherence to uniform distance or driving time between sites.

Facilities should be designed to accommodate the needs of older persons and persons with disabilities. Further information concerning rest area design is presented in the AASHTO *Guide for Development of Rest Areas on Major Arterials and Freeways* (*2*).

3.6.3 Lighting

Lighting may reduce nighttime crashes on a highway or street and improve the ease and comfort of operation thereon. Statistics indicate that nighttime crash rates are higher than daytime crash rates. To a large extent, this may be attributed to reduced visibility at night. There is evidence that in urban and suburban areas, where there are concentrations of pedestrians and roadside intersectional interferences, fixed-source lighting tends to reduce crashes. Lighting of rural highways may be desirable, but the need for it is much less than on streets and highways in urban areas. The general consensus is that lighting of rural highways is seldom justified except in certain critical areas, such as interchanges, intersections, railroad grade crossings, long or narrow bridges, tunnels, sharp curves, and areas where roadside interferences are present. Most modern rural highways should be designed with an open cross section and horizontal and vertical alignment of a fairly high type. Accordingly, they offer an opportunity for near maximum use of vehicle headlights, resulting in reduced justification for fixed highway lighting.

On freeways where there are no pedestrians, roadside entrances, or other intersections at grade, and where rights-of-way are relatively wide, the justification for lighting differs from that of non-controlled streets and highways. The AASHTO *Roadway Lighting Design Guide* (*4*) was prepared to aid in the selection of sections of freeways, highways, and streets for which fixed-source lighting may be warranted, and to present design guide values for their illumination. This guide also contains a section on the lighting of tunnels and underpasses. A primary source of design information for lighting are Illuminating Engineering Society of North America (IESNA) publications, including ANSI/IESNA RP-8, *American National Standard Practice for Roadway Lighting* (*56*); ANSI/IESNA RP-22, *American National Standard Practice for Tunnel Lighting* (*57*); IESNA DG-19, *Design Guide for Roundabout Lighting* (*58*); and IESNA DG-23, *Design Guide for Toll Plaza Lighting* (*59*).

Whether or not rural at-grade intersections should be lighted depends on the layout and the traffic volumes involved. Intersections that do not have channelization are frequently left unlighted. On the other hand, intersections with substantial channelization, particularly multi-road layouts and those designed on a broad scale, are often lighted. It is especially desirable to illuminate large-scale channelized intersections and roundabouts. Because of the sharp curvatures, little of such intersections is within the lateral range of headlights, and the headlights of other vehicles are a hindrance rather than an aid because of the variety of directions and turning movements. There is need to obtain a reduction in the speed of vehicles approaching some intersections. The indication of this need should be definite and visible at a distance from the intersection that is beyond the range of headlights. Illumination of the intersection with fixed-source lighting accomplishes this.

At interchanges it also is desirable, and sometimes essential, to provide fixed-source lighting. Drivers should be able to see not only the road ahead, but also the entire turning roadway area to properly discern the paths to be followed. They should also see all other vehicles that may influence their own behavior. Without lighting, there may be a noticeable decrease in the usefulness of the interchange at night; there would be more cars slowing down and moving with uncertainty at night than during daylight hours. Consideration should be given to improving visibility at night by roadway lighting (or reflectorizing devices) the parts of grade separation structures that particularly should be avoided by motorists, such as curbs, piers, and abutments. The greater the volume of traffic, particularly turning traffic, the more important the fixed-source lighting at interchanges becomes. Illumination should also be considered on those sections of major highways where there are turning movements to and from roadside development.

Floodlighting or highway lighting may be desirable at railroad-highway grade crossings when there are nighttime movements of trains. In some cases, such treatments may apply also to crossings operated with flashing signals, or gates, or both.

Tunnels, toll plazas, and movable bridges are nearly always lighted, as are bridges of substantial length in urban and suburban areas. It is questionable whether the cost of lighting long bridges in rural areas is justified or desirable.

To minimize the effect of glare and to provide the most economical lighting installation, luminaires are mounted at heights of at least 9 m [30 ft]. Lighting uniformity is improved with higher mounting heights, and in most cases, mounting heights of 10 to 15 m [35 to 50 ft] are usually preferable. High-mast lighting—special luminaires on masts of 30 m [100 ft] or greater—is used to light large highway areas such as interchanges and rest areas. This lighting furnishes a uniform light distribution over the whole area and may provide alignment guidance. However, it also has a disadvantage in that the visual impact on the surrounding community from scattered light is increased.

Luminaire supports (poles) should be placed outside the roadside clear zones whenever practical. The appropriate clear zone dimensions for the various functional classifications will be found in the discussion of roadside design in Section 4.6. Where poles are located within the clear zone, regardless of distances from the traveled way, they should be designed to have a suitable impact attenuation feature; normally, a breakaway design is used. Breakaway poles should not be used on streets in densely developed areas, particularly with sidewalks. When struck, these poles could interfere with pedestrians and cause damage to adjacent buildings. Because of lower speeds and parked vehicles, there is much less chance of injuries to vehicle occupants from striking fixed poles on a street as compared to a highway. Poles should not be erected along the outside of curves on ramps where they are more susceptible to being struck. Poles located behind longitudinal barriers (installed for other purposes) should be offset sufficiently to allow for deflection of the longitudinal barriers under impact.

On a divided highway or street, luminaire supports may be located either in the median or on the right side of the roadway. Where luminaire supports are located on the right side of the roadway, the light source is usually closer to the more heavily used traffic lanes. However, with median installation, the cost is generally lower and illumination is greater on the high-speed lanes. For median installations, dual-mast arms should be used, for which 12 to 15 m [40 to 50 ft] mounting heights are favored. For further information, refer to the AASHTO *Roadside Design Guide* (*8*).

Where highway lighting is being considered for future installation, considerable savings can be achieved through design and installation of necessary conduits under roadways and curbs as part of initial construction.

Highway lighting for freeways is directly associated with the type and location of highway signs. For full effectiveness, the two should be designed jointly.

3.6.4 Utilities

Highway and street improvements, whether upgraded within the existing right-of-way or entirely on new right-of-way, generally entail adjustment of utility facilities. Utilities generally have little effect on the geometric design of the highway or street. However, full consideration, reflecting sound engineering principles and economic factors, should be given to measures needed to preserve and protect the integrity and visual quality of the highway or street, its maintenance efficiency, and the safety of traffic. The costs of utility adjustments vary considerably because of the large number of companies, the type and complexity of the facility, and the degree of involvement with the improvement. Depending on the location of a project, the utilities involved could include (1) sanitary sewers; (2) water supply lines; (3) oil, gas, and petroleum product pipelines; (4) overhead and underground power and communications lines including fiber optic cable; (5) cable television; (6) wireless communication towers; (7) drainage and irrigation lines; (8) heating mains; and (9) special tunnels for building connections.

General

Utility lines should be located to minimize need for later adjustment, to accommodate future highway or street improvements, and to permit servicing such lines with minimum interference to traffic.

Longitudinal installation should be located on uniform alignment as near as practical to the right-of-way line so as to not interfere with traffic operation and to preserve space for future highway or street improvements or other utility installations. Underground utilities should be placed to allow above ground utilities to be as close to the right-of-way line as practical. Also to the extent practical, utilities along freeways should be constructed so they can be serviced from outside the controlled access lines.

To the extent practical, utility line crossings of the highway should cross on a line generally normal to the highway alignment. Those utility crossings that are more likely to need future servicing should be encased or installed in tunnels to permit servicing without disrupting the traffic flow.

The horizontal and vertical location of utility lines within the highway right-of-way limits should conform to the clear roadside policies applicable for the system, type of highway or street, and specific conditions for the particular section involved. Utility facilities on highway and street rights-of-way should be located well away from the traveled way and should be designed so they are not roadside obstacles. The clear roadside dimension to be maintained for a specific functional classification is discussed in Section 4.6 on "Roadside Design."

Sometimes attachment of utility facilities to highway structures, such as bridges, is a practical arrangement and may be authorized. Where it is practical to locate utility lines elsewhere, attachment to bridge structures should be avoided.

On new installations or adjustments to existing utility lines, provision should be made for known or planned expansion of the utility facilities, particularly those located underground or attached to bridges.

All utility installations on, over, or under highway or street right-of-way and attached structures should be of durable materials designed for long service-life expectancy, relatively free from routine servicing and maintenance, and meet or exceed the applicable industry codes or specifications.

Utilities that are to cross or otherwise occupy the right-of-way of rural or urban freeways should conform to the AASHTO *Policy on the Accommodation of Utilities within Freeway Right-of-Way* (*6*). Those on non-controlled access highways and streets should conform to the AASHTO *Guide for Accommodating Utilities within Highway Right-of-Way* (*5*).

Rural

On new construction, no utility should be situated under any part of the roadway, except where the utility crosses the highway.

Normally, no poles should be located in the median of divided highways. Utility poles, vent standpipes, and other aboveground utility appurtenances that may be struck by vehicles that run off the road should not be placed within the highway clear zone as discussed in Section 4.6.1. The AASHTO *Roadside Design Guide* (*8*) discusses clear-zone widths and may be used as a reference to determine appropriate widths for freeways, rural arterials, and high-speed rural collectors. For low-speed rural collectors and rural local roads, except for very low-volume local roads with ADTs less than or equal to 400 vehicles per day, a minimum clear zone width of 2 to 3 m [7 to 10 ft] is desirable.

Urban

Because of restricted space in most metropolitan areas, special consideration should be given in the initial design to the potential for joint usage of the right-of-way that is consistent with the primary function of the highway or street.

Appurtenances to underground installations, such as vents, drains, markers, manholes, and shutoffs, should be located so as not to be a roadside obstacle, not to interfere with highway or street maintenance activities, and not to be concealed by vegetation. Preferably they should be located near the right-of-way line.

Where there are curbed sections, utilities should be located in the border areas between the curb and sidewalk, at least 0.5 m [1.5 ft] behind the face of the curb, and where practical, above ground utilities should be behind the sidewalk. Where shoulders are provided rather than curbs, a clear zone commensurate with rural conditions should be provided.

Existing development and limited right-of-way widths may preclude location of some or all utility facilities outside the roadway of the street or highway. Under some conditions, it may be appropriate to reserve the area outside the roadway exclusively for the use of overhead lines with all other utilities located under the roadway, and in some instances the location of all the facilities under the roadway may be appropriate. Location of utilities under the roadway is an exception to the stated policy and as such needs special consideration and treatment. Accommodation of these facilities under the roadway should be accomplished in a manner that will have a minimum adverse effect on traffic as a result of future utility service and maintenance activities.

3.6.5 Traffic Control Devices

Traffic Signs, Pavement Markings, and Traffic Signals

Traffic signs, pavement markings, and traffic signals are directly related to, and complement, the design of highways and streets. They are critical features of traffic control and operation that the designer considers in the geometric layout of such a facility. Traffic control devices should be designed concurrently with the geometrics. The potential for future operational efficiency can be significantly enhanced if signs, markings, and signals are treated as an integral part of design.

The extent to which traffic control devices are used depends on the traffic volume, the type of facility, and the extent of traffic control appropriate for safe and efficient operation. Arterial highways are usually numbered routes of fairly high type and have relatively high traffic volumes. On such highways, signs and markings are employed extensively and traffic signals are often employed in urban areas. Collector and local roads and streets usually have lower volumes and speeds and therefore typically need fewer traffic control devices. The geometric design of the facility should be supplemented by effective signing, markings, and signals as a means of informing, warning, and controlling users during day and night operations and under a variety of environmental conditions. Signing, marking, and signal plans should be coordinated with horizontal and vertical alignment, sight distance obstructions, operational speeds and maneuvers, and other applicable items before completion of design. For requirements and guidance concerning design, location, and application of signs and markings, refer to the MUTCD (*22*).

Traffic control signals for vehicles, pedestrians, and bicycles are devices that control crossing or merging traffic by assigning the right-of-way to various movements for certain intervals of time. They are one of the key elements in the function of many urban streets and some rural intersections. For this reason, the planned traffic signal design and operation for each intersection of a facility should be integrated with the geometric design features to provide optimum operational efficiency. Careful consideration should be given in design to intersection and access locations, horizontal and vertical curvature with respect to signal visibility, pedestrian and bicycle needs (including accommodation of pedestrians with disabilities), and the geometric layout for effective signal operation including signal phasing, timing, and coordination. In addition to the initial installation, potential future signal locations and needs should be considered in the design process. The design of traffic signal devices and warrants for their use are provided in the MUTCD (*22*).

Because supports for highway signs and signals have the potential of being struck by motorists, they should be placed on structures outside the desired clear zone or behind traffic barriers placed for other reasons. If these measures are not practical, the supports should be breakaway or, for overhead sign and signal supports, shielded by appropriate traffic barriers. The AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (*3*) provides the criteria for breakaway sign supports. Likewise, supports should not be placed in such a way that they restrict pedestrian traffic on adjacent sidewalks. Sign supports on sidewalks can severely impact pedestrians with vision impairments and are obstacles to all pedestrians. See Section 4.17 on "Pedestrian Facilities" for details and references.

The number and arrangement of lanes are key to efficient 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 geometric designs should be a complete analysis of current and future traffic demand, including pedestrian, bicycle, and transit users. The need for 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 need for obtaining any additional right-of-way needed. Along a highway or street with a number of signalized intersections, the locations where turns will, or will not, be accommodated should also be examined to permit optimal traffic signal coordination.

Intelligent Transportation Systems

The use of Intelligent Transportation Systems (ITS) on the highway and street system continues to grow in coverage and diversity of technology and applications. In urban areas, traditional ITS applications such as traffic signals and more complex advanced traffic management systems (ATMS) and Advanced Traveler Information Systems (ATIS) are growing in usage and complexity. All of these systems are increasing the number of devices on arterial and sometimes collector roadways. These devices include closed-circuit television cameras, traffic speed and density detectors, dynamic message signs, ramp control signals, transit priority signals, tolling systems, and other types of advanced monitoring and management devices. The communications system infrastructure that connects, controls, and monitors these systems is also an important element of the ITS infrastructure that should be considered in the geometric design process. It is important that the designer identify the existing and planned applications of ITS technologies and their supporting infrastructure elements within the highway and street network to create geometric designs that allow for their effective operation and appropriate physical placement. Most transportation agencies have developed ITS device and infrastructure standards and specifications that can be used in the design process.

3.6.6 Traffic Management Plans for Construction

Maintenance of traffic during construction should be carefully planned and executed (*21*). Although it is often better to provide detours, this is frequently impractical so traffic flow usually is maintained through the construction area. Sometimes traffic lanes are closed, shifted, or encroached upon in order to undertake construction. When this occurs, designs for traffic control should minimize the effect on traffic operations by minimizing the frequency or duration of interference with normal traffic flow. The development of traffic control plans is an essential part of the overall project design and may affect the design of the facility itself. The traffic control plan depends on the nature and scope of the improvement, volumes of traffic, highway or street pattern, and capacities of available highways or streets. A well-thought-out and carefully developed plan for the movement of traffic through a work zone will contribute significantly to the safe and efficient flow of traffic as well as the reduced potential for injury to the construction forces. It is desirable that such plans have some built-in flexibility to accommodate unforeseen changes in work schedule, delays, or traffic patterns.

The goal of any traffic control plan should be to effectively guide vehicle, bicycle, and pedestrian traffic, including persons with disabilities, through or around construction areas. Worker access to the construction area should also be provided. The traffic control plan should incorporate geometrics and traffic control devices as similar as practical to those for normal operating situations, while providing room for the contractor to work effectively. Policies for the use and application of signs and other traffic control devices when highway construction occurs are set forth in the MUTCD (*22*). It cannot be emphasized too strongly that the MUTCD (*22*) principles should be applied and a plan developed for the particular type of work performed.

Adequate advance warning and sufficient follow-up information should be provided to drivers to prepare them for the changed operating conditions in construction areas. The distance that such signing should be located in advance of the work zone varies with the speed on the affected facility. Size of signs may vary depending on the need for greater legibility and emphasis or the type of highway. Construction operations frequently create the need for adjustments in traffic patterns including the shifting of lanes. The minimum taper length for lane transitions in construction areas can be computed by a formula found in the MUTCD (*22*). Various configurations are illustrated in the MUTCD (*22*) and should be used in developing traffic control plans.

The stopping of traffic by a flagger or any other means should be avoided wherever practical. Designs that provide for constant movement around an obstruction in the roadway, even if it is slow, are more acceptable and are less irritating to drivers than designs that require them to stop.

When construction operations are scheduled to take place adjacent to passing traffic, a clear zone should be included in the traffic control plans, wherever practical, between the work space and the passing traffic. Under certain conditions, a positive barrier is justified.

Traffic operational considerations for the design of a detour are speed, capacity, travel distance, and reduced potential for crashes. The speed for a detour may be less than that on the facility being improved but should be high enough so as not to affect the capacity. When an existing highway or street is used as a detour, higher volumes result and it may be appropriate to increase the capacity of such a route in advance. The capacity is generally increased by eliminating troublesome turning movements, rerouting transit vehicles and trucks, banning parking, adopting and enforcing a loading/unloading ban during peak hours, eliminating or adjusting certain transit stops, coordinating signal timing, and sometimes physically widening the traveled way. An effective means of increasing capacity is by instituting a one-way detour system, coupled with parking restrictions. A detour plan is tested by comparing the traffic volumes expected to use the rearranged plan to the calculated capacity of the detour system.

The roadway near construction access points should be well lighted and delineated. Channelization of traffic should be accomplished by the use of signing on yielding supports, pavement markings, and barricades.

Construction areas, detours, and temporary connections often include geometric features and roadway environments that may need more caution and alertness than is normally expected of drivers. Care in the layout of these areas, in the use of delineation and warning devices, and in the establishment of areas for contractor operations is appropriate to reduce the potential for crashes involving both motorists and workers. Items that should be considered in developing traffic control plans include the following:

- Diversion and detour alignments to allow traffic to pass smoothly around the work zones. The surface of the traveled way, whether located within the construction area or on a detour, should be maintained in a condition that will permit the effective movement of traffic at a reasonable speed. The impacts of diverted traffic on other highways, streets, and intersections should be considered.

- Adequate tapers for lane drops or where traffic is shifted laterally. Appropriate values for taper lengths can be found in the MUTCD (*22*).

- In urban areas, diversion provisions for all existing pedestrian flows. The selected diversion paths should include crosswalks with curb ramps, adequate width, a smooth riding surface, wayfinding, and, as appropriate, barricades to accommodate persons with disabilities.

- Adequate traffic control devices and pavement markings for both daytime and nighttime effectiveness, including specifying temporary marking materials that can be removed when traffic-lane patterns change.

- Roadway illumination and warning lights where justified. Steady burning lights are used to delineate a continuous travel path through or around a work zone. The very short "on" time of flashing lights does not enable motorists to focus on the light and make a depth-perception estimate. The use of flashers should be limited to marking a single object or condition, marking the start of a section using steady burn lights, and for use with traffic control signs.
- The location of cones, delineators, drums, barriers, or barricades to channelized traffic, when special conditions exist or if not shown in the standard plans.
- Policies concerning the removal of signs and markings from the job site, when they are no longer needed, if not provided for in the specifications.
- Except in extenuating circumstances, the removal of contractor equipment completely off the roadways, medians, and shoulders at night, on weekends, and whenever equipment is not in operation. In those instances where such removal is not practical, appropriate signing, lighting, barricades, barriers, and similar devices to protect the motorist from collision with the equipment should be specified. The storage of hazardous materials, however, should not be permitted on roadways, medians, or shoulders near the flow of traffic.
- A limitation in the plans or specifications on parking of employees' private vehicles in those areas on the project that may interfere with workers or with through traffic.

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