AWS D1.2/D1.2M[3]: *Structural Welding Code—Aluminum*, Sixth ed., 2014. American Welding Society, Miami, FL.

AWS D1.4/D1.4M[4]: *Structural Welding Code—Reinforcing Steel*, Seventh ed., 2011. American Welding Society, Miami, FL.

IBC: *2015 International Building Code* (without supplements). International Code Council, Inc., Falls Church, VA.

NDS[5]: *National Design Specification for Wood Construction ASD/LRFD*, 2015 ed., and *National Design Specification Supplement*, *Design Values for Wood Construction*, 2015 ed. American Wood Council, Washington, DC.

OSHA 1910: *Occupational Safety and Health Standards* (U.S. Federal version) Subpart A, General, 1910.1–1910.9, with Appendix A to 1910.7; Subpart D, Walking-Working Surfaces, 1910.21–1910.30; Subpart F, Powered Platforms, Manlifts, and Vehicle-Mounted Work Platforms, 1910.66–1910.68, with Appendix A–Appendix D to 1910.66. U.S. Department of Labor, Washington, DC.

OSHA 1926: *Occupational Safety and Health Regulations for the Construction Industry* (U.S. Federal version) Subpart E, Personal Protective and Life Saving Equipment, 1926.95–1926.107; Subpart M, Fall Protection, 1926.500–1926.503, App. A–E; Subpart Q, Concrete and Masonry Construction, 1926.700–1926.706, with App. A; and Subpart R, Steel Erection, 1926.750–1926.761, with App. A–H. U.S. Department of Labor, Washington, DC.

PCI: *PCI Design Handbook: Precast and Prestressed Concrete*, Seventh ed., 2010. Precast/Prestressed Concrete Institute, Chicago, IL.

TRANSPORTATION DESIGN STANDARDS

AASHTO *GDPS*: *AASHTO Guide for Design of Pavement Structures* (GDPS-4-M), 1993, and 1998 supplement. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO *Green Book*: *A Policy on Geometric Design of Highways and Streets*, ~~Sixth~~ ed., ~~2011~~ (including ~~November 2013~~ errata). American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO: *Guide for the Planning, Design, and Operation of Pedestrian Facilities*, First ed., 2004. American Association of State Highway and Transportation Officials, Washington, DC.

HSM: *Highway Safety Manual*, First ed., 2010 vols. 1-3 (including September 2010, Febrary 2012, and March 2016 errata). American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO *MEPDG*: *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice*, Second ed., 2015. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO: *Roadside Design Guide*, Fourth ed., 2011 (including February 2012 and July 2015 errata). American Association of State Highway and Transportation Officials, Washington, DC.

AI: *The Asphalt Handbook* (MS-4), Seventh ed., 2007. Asphalt Institute, Lexington, KY.

FHWA: *Hydraulic Design of Highway Culverts*, Hydraulic Design Series no. 5, Publication no. FHWA-HIF-12-026, Third ed., 2012. U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

HCM: *Highway Capacity Manual*, Sixth ed., Transportation Research Board, National Research Council, Washington, DC.

MUTCD: *Manual on Uniform Traffic Control Devices*, 2009 (including Revisions 1 and 2, May 2012). U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

PCA: *Design and Control of Concrete Mixtures*, Sixteenth ed., 2016. Portland Cement Association, Skokie, IL.

[3]See Ftn. 2.
[4]See Ftn. 2.
[5]Only the ASD method may be used for wood design on the structural depth exam.

Table 73.1 *Abbreviations*

abbreviation	meaning
ln	lane
km/h	kilometers per hour
mph	miles per hour
p	people, person, or pedestrian
pc	passenger car
pcph	passenger cars per hour
pcphg	passenger cars per hour of green signal
pcphpl	passenger cars per hour per lane
pcphgpl	passenger cars per hour of green signal per lane
pc/km/ln	passenger cars per kilometer per lane
pcpmpl	passenger cars per mile per lane
ped	pedestrian
pers	person
veh	vehicle
vph	vehicles per hour
veh/km	vehicles per kilometer
vph/lane	vehicles per hour per lane
vpk	vehicles per kilometer
vpm	vehicles per mile

3. FACILITIES TERMINOLOGY

Although common usage does not always distinguish between highways, freeways, and other types of roadways, the major traffic/transportation references are more specific. (See Table 73.2.) For example, "vehicle" and "car" have different meanings. "Vehicle" encompasses trucks, buses, and recreational vehicles, as well as passenger cars.

A *freeway* is a divided corridor with at least two lanes in each direction that operates in an *uninterrupted flow* mode (i.e., without *fixed elements* such as signals, stop signs, and at-grade intersections). *Access points* are limited to ramp locations. Since grades, curves, and other features can change along a freeway, performance measures (e.g., capacity) are evaluated along shorter *freeway segments*.

Multilane and two-lane *highways*, on the other hand, contain some fixed elements and access points from at-grade intersections, though relatively uninterrupted flow can occur if signal spacing is greater than 2 mi (3.2 km). Where signal spacing is less than 2 mi (3.2 km), the roadway is classified as an *urban street* (or *arterial*), and flow is considered to be interrupted. An urban street has a significant amount of driveway access, while an arterial does not. *Divided highways* have separate roadbeds for the opposing directions, whereas *undivided highways* do not.

Smaller roadways are classified as *local roads* and *streets*. All roadways can be classified as *urban*, *suburban*, or *rural*, depending on the surrounding population density. *Urban areas* have populations greater than 5000. *Rural areas* are outside the boundaries of urban areas. Urban and rural transportation facilities are further described in AASHTO *Green Book* Sec. ~~1.3~~.

Table 73.2 *General Functional Classifications of Roadways**

road designation	ADT (vpd)
local road	2000 or less
collector road	2000–12,000
arterial/urban road	12,000–40,000
freeway	30,000 and above

*Classifications can also be established based on percentages of total length and travel volume.

4. DESIGN VEHICLES

Standard *design vehicles* have been established to ensure that geometric features will accommodate all commonly sized vehicles. Standard design vehicles are given specific designations in the AASHTO *Green Book*, as listed in Table 73.3. The AASHTO *Green Book* also contains information on *wheelbase* and *turning radius*. (See AASHTO *Green Book* Fig. 2-1 through Fig. 2-23.)

5. LEVELS OF SERVICE

A user's quality of service through or over a specific facility (e.g., over a highway, through an intersection, across a crosswalk) is classified by a *level of service* (LOS). Levels of service are designated A through F. Level A represents unimpeded flow, which is ideal but only possible when the volume of traffic is small. Level F represents a highly impeded, packed condition. Generally, level E will have the maximum flow rate (i.e., capacity).

The desired design condition is generally between levels A and F. Economic considerations favor higher volumes and more obstructed levels of service. However, political considerations favor less obstructed levels of service. The parameter used to define the level of service varies with the type of facility, as listed in Table 73.4.

Since levels of service can vary considerably during an hour, capacity and LOS evaluations focus on the peak 15 min of flow.

6. SPEED PARAMETERS

Several measures of vehicle speed are used in highway design and capacity calculations. Most measures will not be needed in every capacity calculation. The *design speed* is the maximum safe speed that can be maintained over a specified section of roadway when conditions are so favorable that the design features of the roadway

Table 73.3 Standard Design Vehicles

designation	symbol*	height (ft)	height (m)	width (ft)	width (m)	length (ft)	length (m)
passenger car	P	4.3	(1.30)	7.0	(2.13)	19.0	(5.79)
single-unit truck	SU-30 (SU-9)	11.0–13.5	(3.35–4.11)	8.0	(2.44)	30.0	(9.14)
~~single-unit bus~~	BUS-40 (BUS-12)	12.0	(3.66)	8.5	(2.59)	40.5	(12.36)
articulated bus	A-BUS	11.0	(3.35)	8.5	(2.59)	60.0	(18.29)
combination trucks							
intermediate semitrailer	WB-40 (WB-12)	13.5	(4.11)	8.0	(2.44)	45.5	(13.87)
"double-bottom" semitrailer or full trailer	WB-67D (WB-20D)	13.5	(4.11)	8.5	(2.59)	72.3	(22.04)
recreational vehicles							
motor home	MH	12.0	(3.66)	8.0	(2.44)	30.0	(9.14)
car and camper trailer	P/T	10.0	(3.05)	8.0	(2.44)	48.7	(14.84)
car and boat trailer	P/B	–	–	8.0	(2.44)	42.0	(12.80)
motor home and boat trailer	MH/B	12.0	(3.66)	8.0	(2.44)	53.0	(16.15)

(Multiply ft by 0.3048 to obtain m.)

*Symbols in parentheses represent the SI designations.

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govern. Most elements of roadway design depend on the design speed. The *legal speed* on a roadway section is often set at approximately the *85th percentile speed*, determined by observation of a sizable sample of vehicles. In suburban and urban areas, the legal speed limit is often influenced by additional considerations such as visibility at intersections, the presence of driveways, parking and pedestrian activity, population density, and other local factors. Typical minimum design speeds are given in Table 73.5.

Table 73.5 Minimum Design Speeds (local and rural roads)

design volumes, ADT	terrain (mph (km/h)) level	rolling	mountainous
≥ 2000	50 (80)	40 (60)	30 (50)
~~1500–2000~~	~~50 (80)~~	~~40 (60)~~	~~30 (50)~~
400~~–1500~~	50 (80)	40 (60)	30 (50)
250–400	40 (60)	30 (50)	20 (30)
50–250	30 (50)	30 (50)	20 (30)
< 50	30 (50)	20 (30)	20 (30)

(Multiply mph by 1.609 to obtain km/h.)

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The *average highway speed*, AHS, is the weighted average of the observed speeds within a highway section based on each subsection's proportional contribution to total mileage. The *running speed* is the speed over a specified section of roadway equal to the distance divided by the running time, or the total time required to travel over the roadway section, disregarding any stationary time. The *average running speed* is the running speed for all traffic equal to the distance summation for all cars divided by the time summation for all cars.

The *average spot speed*, also known as the *time mean speed*, is the arithmetic mean of the instantaneous speeds of all cars at a particular point. The *average travel speed* is the speed over a specified section of highway, including operational delays such as stops for traffic signals. The *free-flow speed* is measured using the mean speed of passenger cars under low to moderate flow conditions (up to 1400 pcphpl). The *operating speed* is the highest overall speed at which a driver can safely travel on a given highway under favorable weather conditions and prevailing traffic conditions. (The term *operating speed*, as used by AASHTO and in previous *HCMs*, is similar to free-flow speed when evaluated at low-volume conditions.) *Space mean speed* in a specific time period is calculated by taking the total distance traveled by all vehicles and dividing by the total of the travel times of all vehicles. *Crawl speed* is the maximum sustained speed that heavy vehicles can maintain on a given extended upgrade.

reciprocal of pedestrian density is *pedestrian space*, A_p (ft^2/ped or m^2/ped), where v_p is the pedestrian flow per unit width (ped/min-ft or ped/min·m).

$$A_p = \frac{S_p}{v_p} \quad [\textit{HCM} \text{ Eq. 24-4}] \qquad 73.38$$

The maximum pedestrian *capacity* in walkways is 23 ped/min-ft, pedestrians per minute per foot of walkway width (75 ped/min-m). This occurs when the space is approximately 5–9 ft^2/ped (0.47–0.84 m^2/ped). Capacity drops significantly as space per pedestrian decreases, and movement effectively stops when space is reduced to 2–4 ft^2/ped (0.19–0.37 m^2/ped).

The *HCM* reports impeded flow starts at 530 ft^2/ped (49 m^2/ped), which is equivalent to 0.5 ped/min-ft (1.6 ped/min·m). These values are taken as the limits for LOS A. Also reported is that jammed flow in platoons starts at 11 ft^2/ped (1 m^2/ped), corresponding to 18 ped/min-ft (59 ped/min·m), which are used as the thresholds for LOS F. A *platoon* is a group of pedestrians walking together in a group.

25. CROSSWALKS

(Capacity analysis of pedestrians and bicycles at crosswalks is covered in *HCM* Chap. 19.)

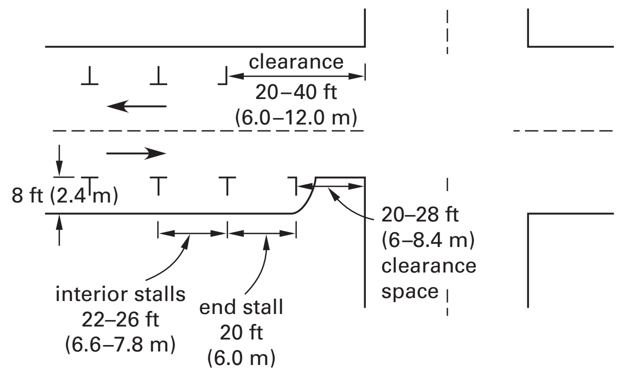
Analysis of pedestrians in *crosswalks* is slightly different than in pure walkways. Walking is affected by signaling, turning vehicles, platooning, and interception of the platoon of pedestrians coming from the opposite side.

26. PARKING

The minimum parallel street parking *stall width* is commonly taken as 7 ft (2.1 m) in a residential area and may range from 8 ft to 11 ft (2.4 m to 3.3 m) in a commercial or industrial area. This width accommodates the vehicle and its separation from the curb. Stalls 7 ft (2.1 m) wide are substandard and should be limited to residential areas and attendant-parked lots. Widths larger than 9 ft (2.7 m) are appropriate in shopping areas where package loading is expected. If the width is specified between 10 ft and 12 ft (3.0 m and 3.7 m), the street parking corridor can be used for delivery trucks or subsequently converted to an extra traffic lane or bicycle path. The minimum length of a parallel street parking stall is 18 ft (5.4 m). In order to accommodate most cars, longer lengths between 20 ft and 26 ft (6.0 m and 7.8 m), may be used.

Figure 73.8 illustrates parallel street parking near an intersection as recommended by the AASHTO *Green Book*. The 20–28 ft (6–8.4 m) clearance from the last stall to the intersection is required to prevent vehicles from using the parking lane for right-turn movements.

Figure 73.8 *AASHTO Green Book Parallel Parking Design*



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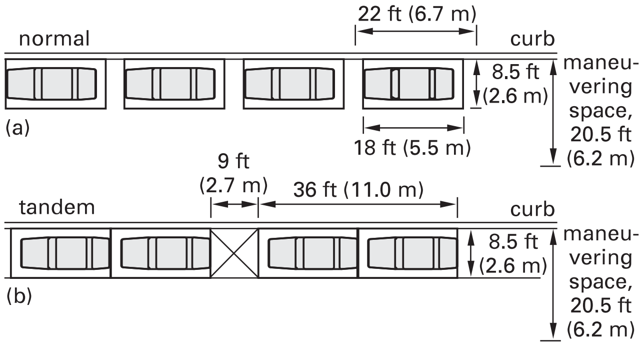
Tandem parking (also known as *double-alternate parking*) provides two shortened stalls placed end to end with a single maneuver zone. With tandem parking, each vehicle will be boxed in in at least one direction. A parking vehicle pulls into the stall space and maneuver zone. The vehicle is removed from the traffic stream in about 4 sec. (With the traditional parallel parking procedure, traffic may halt for as much as 30 sec or more while a vehicle attempts to back into the parking space.) The original (1970) tandem design required 56 ft (17.1 m): two 20 ft (6.1 m) stalls and a 16 ft (4.9 m) maneuver zone. With modern smaller cars, the required space can be reduced to 45–50 ft (13.7–15.3 m): two 18 ft (5.5 m) stalls and a 9–14 ft (2.7–4.3 m) maneuver zone. Other dimensions may work. Typical tandem parking geometry is given in Fig. 73.9.

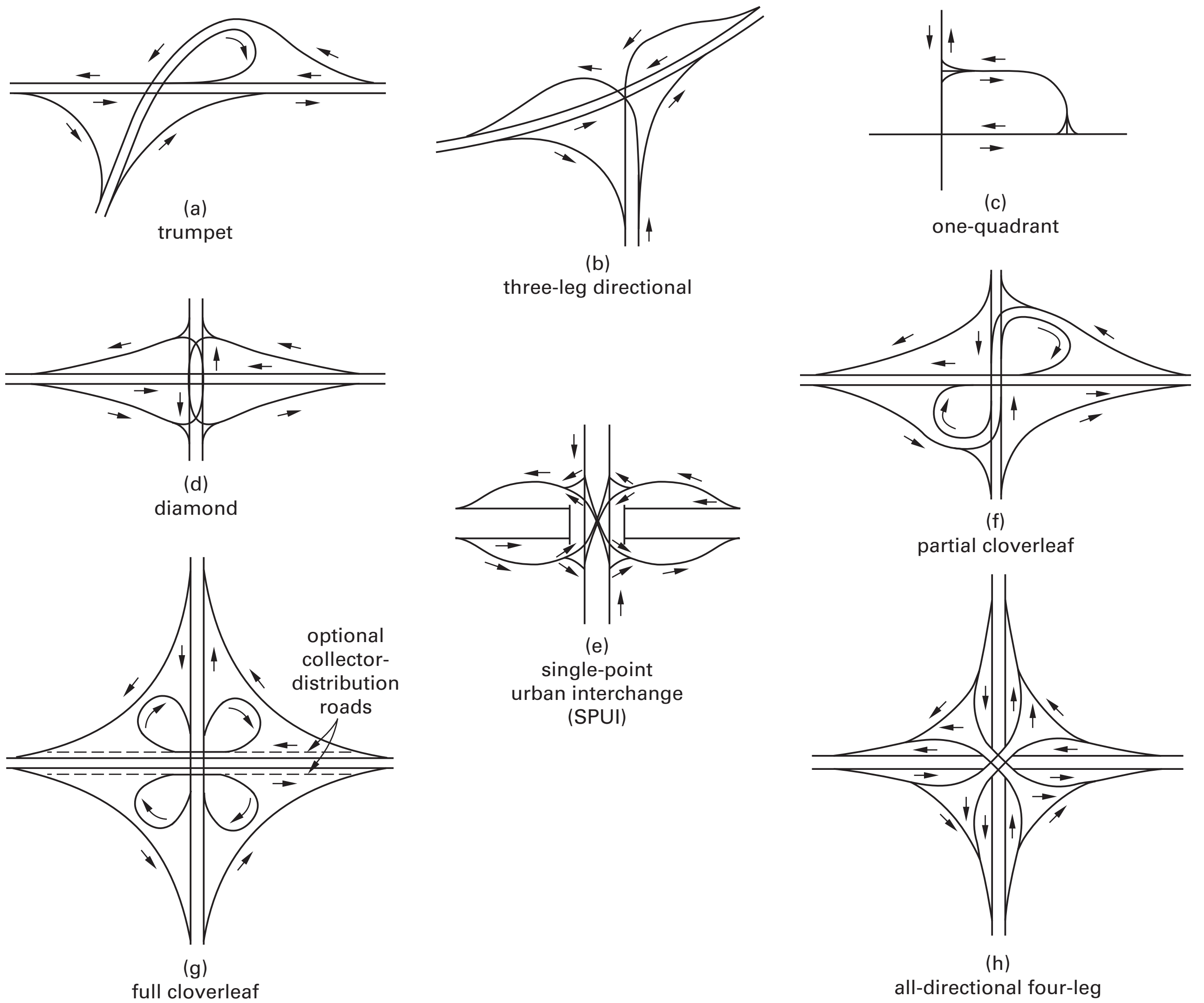
Figure 73.9 *Typical Tandem Curb Parking Geometry*



Diagonal parking (*angle parking*) can be specified with angles to the curb of 45°, 60°, 75°, and 90°. The effects of diagonal parking on lane width can be determined from trigonometry. The significant disadvantage of impaired vision when backing up should be considered when designing angle parking.

In designing *parking lots*, the maximum capacity of the lot can be calculated by dividing the gross lot area by the minimum area per car (e.g., 280–320 ft^2 (26–30 m^2) depending on the types of cars in the community).

Figure 73.11 *Interchange Types*



The main advantage of SPUIs is that they require only a narrow right-of-way, thereby reducing land acquisition cost. The main disadvantage, as with all traffic bridges, is a high construction cost. There are additional geometric design features that require careful consideration, such as the elliptical left-hand turning path, pedestrian accommodations, and the difficulty in accommodating freeways approaching with high skew angles (e.g., more than 30°).

28. WEAVING AREAS

(Analysis and design procedures for weaving areas are covered in *HCM* Chap. 13.)

Weaving is the crossing of at least two traffic streams traveling in the same general direction along a length of highway without traffic control. In the freeway segment shown in Fig. 73.13, flows A–D and B–C cross the paths of other traveling vehicles, so these flows are the segment's *weaving movements*. Flows A–C and B–D do not cross any other vehicles' paths, so they are the segment's *nonweaving movements*.

Weaving is an issue that must be considered in interchange selection, and interchanges without weaving are favored over interchanges with weaving. Weaving areas require increased lane-change maneuvers and result in increased traffic turbulence. Making a weaving segment longer allows for more time for lane changes. Under demand conditions, longer weaving segments increase capacity and decrease traffic density and turbulence.

Figure 73.12 *Adaptability of Interchanges on Freeways as Related to Types of Intersecting Facilities*

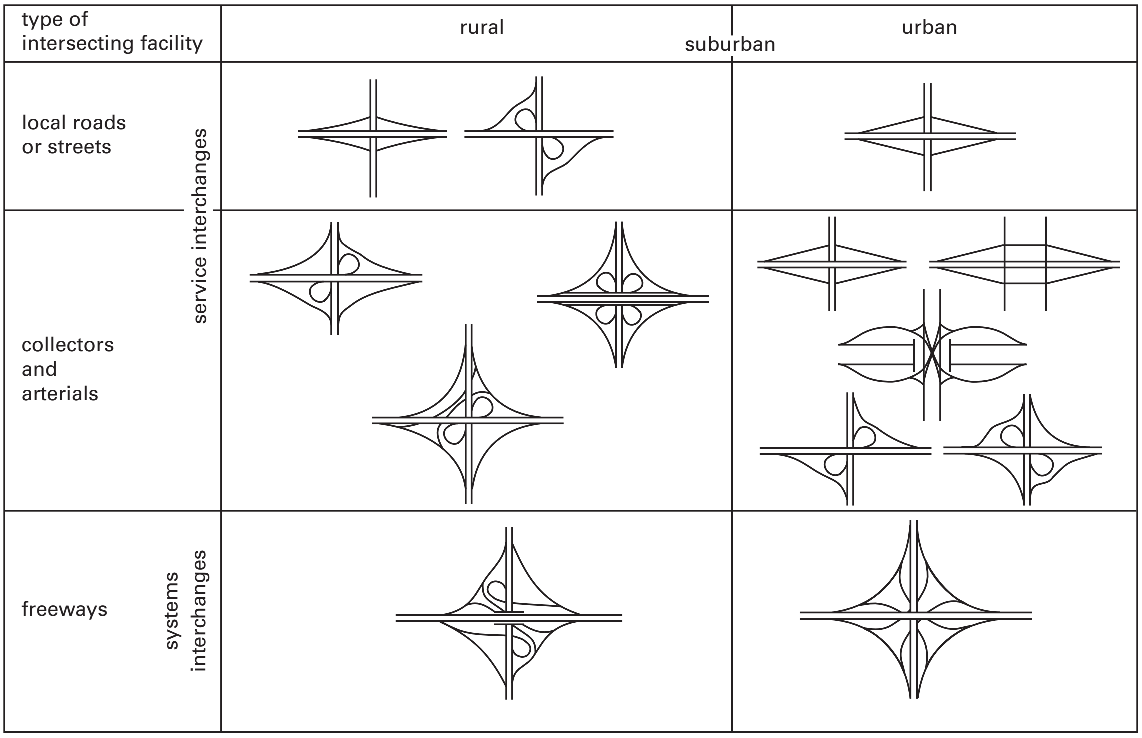


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Figure 73.13 *Freeway Weaving Segment*

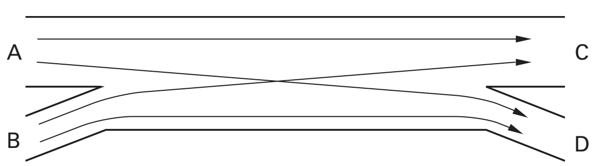

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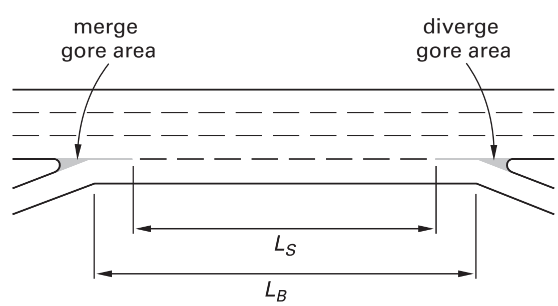
Figure 73.14 *Lengths of a Weaving Segment*



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The operating characteristics of a weaving segment are affected by the length, width, and configuration of the weaving segment. The *weaving segment length*[3] is the distance between the merge and diverge segments. There are two measures of length used in the *HCM*, as shown in Fig. 73.14. The *short length*, L_S, is the distance between the end points of any barrier markings (i.e., solid white lines) that discourage or prohibit lane changing. The *base length*, L_B, is the distance between *gore areas* where the left edge of the ramp lane and the right edge of the freeway lane meet. Where no solid white lines are present, the two lengths are the same (i.e., $L_S = L_B$).

The *weaving segment width* is measured as the number of continuous lanes within the merging and diverging (i.e., entry and exit) gore areas. It is primarily controlled by the number of lanes on the entry and exit legs and the weaving segment configuration.

The *weaving segment configuration* refers to the relative placement and number of entry and exit lanes for a roadway section. Configuration is based on the number

[3]Previous editions of the *HCM* tied definitions of weaving segments to cloverleaf interchanges as most existing weaving segments were part of such interchanges. The *HCM* has replaced such definitions with a more general definition of length to reflect the fact that newer weaving segments occur in a variety of situations and designs.

(b) Since the skid marks were only 90 m long, the vehicle would have skidded 175.1 m − 90 m = 85.1 m further. Solve Eq. 75.25 for the speed.

$$85.1\ \text{m} = \frac{v_{\text{km/h}}^2}{254(f+G)} = \frac{v^2}{(254)(0.35+0.03)}$$
$$v = 90.6\ \text{km/h}$$

Customary U.S. Solution

(a) Use Eq. 75.25. The grade is positive since the car skids uphill.

$$s_b = \frac{v_{\text{mph}}^2}{30(f+G)} = \frac{\left(80\ \frac{\text{mi}}{\text{hr}}\right)^2}{(30)(0.35+0.03)}$$
$$= 561.4\ \text{ft}$$

(b) Since the skid marks were only 300 ft long, the vehicle would have skidded 561.4 ft − 300 ft = 261.4 ft further. Solve Eq. 75.25 for the speed.

$$261.4\ \text{ft} = \frac{v_{\text{mph}}^2}{30(f+G)} = \frac{v^2}{(30)(0.35+0.03)}$$
$$v = 54.6\ \text{mph}$$

10. SPEED DEGRADATION ON UPHILL GRADES

Most modern passenger cars traveling on highways are capable of negotiating uphill grades of 4–5% without speed decreases below their initial level-highway speeds. (Older cars with high mass-to-power ratios and some smaller-sized "economy" vehicles may experience speed decreases.)

Heavy trucks experience greater speed degradations than passenger cars. The primary variables that affect actual speed decreases include the grade steepness, the grade length, and the truck's mass-to-power ratio. *Mass-to-power ratios* are commonly stated in pounds per horsepower (lbm/hp) and kilograms per kilowatt (kg/kW). (Multiply lbm/hp by 0.6083 to obtain kg/kW.) AASHTO *Green Book* Chap. 3 contains simple graphs of speed decreases for "heavy trucks" with mass-to-power ratios of 200 lbm/hp (120 kg/kW) and recreational vehicles entering ascending grades at 55 mph (88.5 km/h).

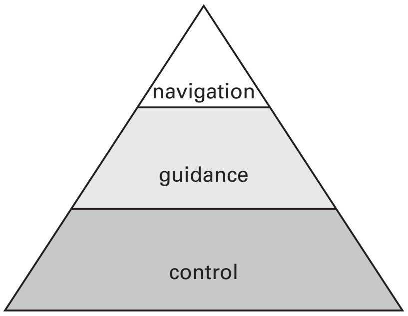
11. FACTORS CONTRIBUTING TO CRASHES

Three types of factors contribute to crashes: human (i.e., physiological and emotional), environmental (including the roadway), and vehicle. *Human factors* include age, judgment, skill, attention, cell phone usage, speed, fatigue, experience, and sobriety. *Roadway/environmental factors* include horizontal geometry, traffic control devices, signs, surface friction (including dry/wet conditions), grade (i.e., steepness), weather, and medical response time. *Vehicle factors* include vehicle design (e.g., anti-lock braking systems and bumper height), vehicle condition (including brakes and tires), and maintenance.

According to a study reported in the HSM, human factors contribute to a crash 93% of the time, followed by roadway factors at 34% and vehicle factors at 13%. As the percentages indicate, the three types of factors interact and overlap in complex ways.

The HSM divides the *driving task* into three hierarchal sub-tasks: control, guidance, and navigation. (See Fig. 75.1.) The *control sub-task* includes maintaining the appropriate speed and staying within the driving lane. The *guidance sub-task* includes safely interacting with other vehicles and following traffic control devices. The *navigation sub-task* includes traveling a path from origin to destination by reading guide signs. Although the navigation sub-task is the most complex, the control sub-task forms the basis of the other sub-tasks.

Figure 75.1 *Driving Task Hierarchy*



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Driver error is a contributing factor in most crashes. Driver error includes errors of judgment, distractions, inattentiveness, weariness, overload of information, and deliberate violation of traffic laws. These errors are categorized into physical, perceptual, and cognitive limitations. In addition to physiological factors such as perception-reaction time, primary factors that affect driving safety and contribute to crashes are drivers' attention and information processing, vision, and speed choice.

Attention and Information Processing

Driver overload occurs when a driver's attention and ability to process information is limited. Overloading makes it difficult for a driver to divide his or her attention between the control, guidance, and navigation sub-tasks. Table 75.2 presents common causes of driver determining the superelevation rate when friction is relied upon to counteract some of the centrifugal force. The sum, $e + f_s$, may be referred to as the *centrifugal factor*.

$$e = \tan\phi = \frac{\mathrm{v}^2}{gR} - f_s \quad \text{[consistent units]} \qquad 79.36$$

If the velocity, v, is expressed in common units, Eq. 79.36 becomes

$$e = \tan\phi = \frac{\mathrm{v}_{\mathrm{km/h}}^2}{127R} - f_s \quad \text{[SI]} \qquad 79.37(a)$$

$$e = \tan\phi = \frac{\mathrm{v}_{\mathrm{mph}}^2}{15R} - f_s \quad \text{[U.S.]} \qquad 79.37(b)$$

For very large values of R, Eq. 79.36 and Eq. 79.37 become negative. However, curves with very large radii do not need to be superelevated. The normal slopes of the terrain and crown limit the lower value of e.

Many studies have been performed to determine values of the *side friction factor*, f_s, and most departments of transportation have their own standards. One methodology is to assume the side friction factor to be 0.16 for speeds less than 30 mph (50 km/h). Equation 79.38 or Eq. 79.39 can be used for higher speeds.

$$f_s = 0.16 - \frac{0.01(\mathrm{v}_{\mathrm{mph}} - 30)}{10} \quad [< 50 \text{ mph}] \qquad 79.38$$

$$f_s = 0.14 - \frac{0.02(\mathrm{v}_{\mathrm{mph}} - 50)}{10} \quad [50 - 70 \text{ mph}] \qquad 79.39$$

Although the sophistication may be unwarranted, the friction factor for sideways slipping, f_s, can be differentiated from the straight-ahead friction factor. For sideways slipping, the friction factor may be referred to as the *side friction factor*, *lateral ratio*, *cornering ratio*, or *unbalanced centrifugal ratio*.

In general, a lower *banking angle* is used in urban areas than in rural areas. (For low-speed urban streets, use of superelevation is optional [AASHTO *Green Book*].) For arterial streets in downtown areas, the maximum superelevation rate is approximately 0.04–0.06. For arterial streets in suburban areas and on freeways where there is no snow or ice, the maximum superelevation rate is approximately 0.08–0.12. For arterial streets and freeways that experience snow and ice, the maximum should be 0.06–0.08.

Since the maximum superelevation rate is approximately 0.08 or 0.12, Eq. 79.35 can be used to calculate the minimum curve radius if the speed is known.

11. SUPERELEVATION TABLES

While theoretical superelevation can be calculated from the design speed and side friction factors, superelevation rate problems are almost always solved by using the AASHTO *Green Book* superelevation graphs (Fig. 3-9 through Fig. 3-13) and tables (Table 3-8 through Table 3-12). This is because by using the AASHTO graphs and tables, one can incorporate all the various considerations affecting superelevation design and eliminate common difficulties that accompany using the theoretical formulas (e.g., not knowing the maximum side friction factor and superelevation rates that become negative when the curve radius is large, leading to an erroneous conclusion that superelevation is not required and side friction alone should be used to resist the lateral force).

AASHTO *Green Book* Table 3-8 through Table 3-12 are used to determine the minimum radius (i.e., the sharpest curve) for any superelevation. When used to determine the sharpest curve that can be traveled without superelevation, the AASHTO tables incorporate an average cross slope rate of 1.5% (i.e., a normal cross slope).

However, in order to use the AASHTO graphs and tables, it is necessary to know the maximum superelevation, $e_{\max}$, that will be achieved in the circular curve. $e_{\max}$ is typically limited by standards established by the transportation agency. The value selected takes into consideration local climate, road materials and conditions, terrain, adjacent land use, and nature of the traffic (e.g., fraction of tractors and other slow-moving vehicles).

Interpolation is not used with AASHTO *Green Book* Table 3-8 through Table 3-12. Instead, when entering the table with a known radius, a superelevation rate corresponding to the nearest smaller radius is used.

Example 79.4

A low speed rural street has a design speed of 25 mph, a curve radius of 525 ft, and a maximum superelevation rate of 8%. What is the superelevation rate according to the AASHTO *Green Book*?

Solution

Use Table 3-10~~b~~. Enter from the top with a design speed of 25 mph, proceed down the 25 mph column to the closest lower curve radius (499 ft in the table), and read the superelevation of 5.0% in the left column.

12. TRANSITIONS TO SUPERELEVATION

Transitions from crowned sections to superelevated sections should be gradual. When a curve is to be superelevated, it is first necessary to establish a rotational axis (often referred to as a "point") on the cross section. This is the axis around which the pavement will be rotated (longitudinally) to gradually change to the specified superelevated cross slope. The rotational axis ("point") is a longitudinal axis parallel to the instantaneous direction of travel. The location of the rotational point varies with the basic characteristics of the typical section. The following guidelines can be used. (See Fig. 79.8.)

1. For two-lane and undivided highways, the axis of rotation is generally at (along) the original crown of the roadway. However, it may also be the edge of the outside or inside lane.

2. On divided highways with relatively wide depressed medians, the axis of rotation can be at the crown of each roadway or at the edge of the lane or shoulder nearest the median of each roadway. Placing the axis at the crown results in median edges at different elevations, but it reduces the elevation differential between extreme pavement edges. If it is likely that the highway will be widened in the future, it will be desirable to rotate the pavement cross-slope about the inside lane or shoulder.

3. On divided highways with narrow raised medians and moderate superelevation rates, the axis of rotation should be at the center of the median. If the combination of pavement width and superelevation rate results in substantial differences between pavement edge elevations, the axis of rotation should be at the edge of the lane or shoulder nearest the median of each roadway. If an at-grade crossing is located on the superelevated curve, the impact of intersecting traffic should be considered in selecting the axis of rotation.

4. On divided highways with concrete median barriers, the median elevation must be the same for both directions of travel. Rotation should occur at the barrier gutter.

For maximum comfort and safety, superelevation should be introduced and removed uniformly over a length adequate for the likely travel speeds. The total length of the *superelevation transition distance* is the sum of the tangent (crown) runout and the superelevation runoff. The following design factors should be considered in designing the superelevation transition distance.

1. *Tangent runout*, T_R, also known as *tangent runoff* and *crown runoff*, is a gradual change from a normal crowned section to a point where the *adverse cross slope* on the outside of the curve has been removed. When the adverse cross slope has been removed, the elevation of the outside pavement edge will be equal to the centerline elevation. The inside pavement edge will be unchanged. The rate of removal is usually the same as the superelevation runoff rate, SRR.

2. *Superelevation runoff*, L, is a gradual change from the end of the tangent runout to a cross section that is fully superelevated. The *superelevation runoff rate* (also known as the *transition rate*), SRR, is the rate at which the normal cross-slope of the roadway is transitioned to the superelevated cross-slope. The superelevation runoff rate is expressed in units of cross-slope elevation per unit width per unit length of traveled roadway.

   For single lanes, a common superelevation runoff rate is 1 ft per foot of width for every 200 ft (60 m) length, expressed as 1:200. For speeds less than 50 mph, or if conditions are restrictive and if sufficient room is not available for a 1:200 transition rate, more abrupt rates may be used.

3. Tangent runout and superelevation runoff distances may be calculated using Eq. 79.40 and Eq. 79.41. w is the lane width, and p is the rate of cross slope.

$$T_R = \frac{wp}{\text{SRR}} \qquad 79.40$$

$$L = \frac{we}{\text{SRR}} \qquad 79.41$$

4. Superelevation runoff rate on a curve may be determined by a *time rule* (e.g., "runoff shall be completed within 4 sec at the design speed") or by a *speed rule* (e.g., "3 ft of runoff for every mph of design speed"), regardless of the initial superelevation. Time and speed rules depend on the specifying agency.

5. On circular curves, the superelevation runoff should be developed 60–90% on the tangent and 40–10% on the curve, with a large majority of state highway agencies using a rule of two-thirds on the tangent and one-third on the curve. This results in two-thirds of the full superelevation at the beginning and ending of the curve. This is a compromise between placing the entire transition on the tangent section, where superelevation is not needed, and placing the transition on the curve, where full superelevation is needed. AASHTO *Green Book* ~~Fig. 3-8~~ gives specific runoff recommendations based on speed and number of rotated lanes.

6. On spiral curves, the superelevation is developed entirely within the length of the spiral.

7. All shoulders should slope away from the traveled lanes. The angular breaks at the pavement edges of the superelevated roadways should be rounded by the insertion of vertical curves. The minimum curve length in feet should be approximately numerically equal to the design speed in miles per hour.

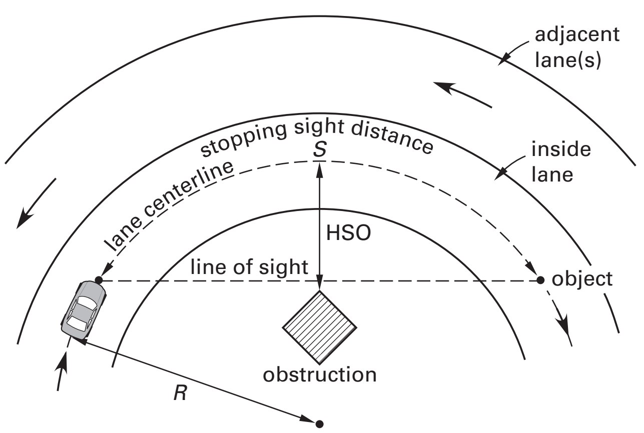
Table 79.2 AASHTO Minimum Stopping Sight Distances on Level Roadways (based on braking distance)[a,b]

design speed		brake reaction distance		braking distance on level		stopping sight distance: calculated		stopping sight distance: design	
(mph)	(km/h)	(ft)	(m)	(ft)	(m)	(ft)	(m)	(ft)	(m)
15	(20)	55.1	(13.9)	21.6	(4.6)	76.7	(18.5)	80	(20)
20	(30)	73.5	(20.9)	38.4	(10.3)	111.9	(31.2)	115	(35)
25	(40)	91.9	(27.8)	60.0	(18.4)	151.9	(46.2)	155	(50)
30	(50)	110.3	(34.8)	86.4	(28.7)	196.7	(63.5)	200	(65)
35	(60)	128.6	(41.7)	117.6	(41.3)	246.2	(83.0)	250	(85)
40	(70)	147.0	(48.7)	153.6	(56.2)	300.6	(104.9)	305	(105)
45	(80)	165.4	(55.6)	194.4	(73.4)	359.8	(129.0)	360	(130)
50	(90)	183.8	(62.6)	240.0	(92.9)	423.8	(155.5)	425	(160)
55	(100)	202.1	(69.5)	290.3	(114.7)	492.4	(184.2)	495	(185)
60	(110)	220.5	(76.5)	345.5	(138.8)	566.0	(215.3)	570	(220)
65	(120)	238.9	(83.4)	405.5	(165.2)	644.4	(248.6)	645	(250)
70	(130)	257.3	(90.4)	470.3	(193.8)	727.6	(284.2)	730	(285)
75	(140)	275.6	(97.3	539.9	(224.8)	815.5	(322.1)	820	(325)
80		294.0		614.3		908.3		910	
85		313.5		693.5		1007.0		1010	

(Multiply ft by 0.3048 to obtain m.)
(Multiply mph by 1.6093 to obtain km/h.)

[a]Brake reaction distance predicated on a time of 2.5 sec; deceleration rate of 11.2 ft/sec^2 (3.4 m/s^2) used to determine calculated sight distance.
[b]Use AASHTO *Green Book* Table 3-2 for roadways on grades.
Adapted from *A Policy on Geometric Design of Highways and Streets*, Table 3-1, copyright © ~~2011~~ by the American Association of Highway and Transportation Officials, Washington, D.C. Used by permission.

Figure 79.9 Horizontal Circular Curve with Obstruction



17. VERTICAL CURVES

Vertical curves are used to change the elevations of highways. Curves that approach higher elevations are known as *crest curves*. *Sag curves* approach lower elevations. Most vertical curves take the shape of an *equal-tangent parabola*. Such curves are symmetrical about the vertex. Since the grades are very small, the actual arc length of the curve is approximately equal to the chord length BVC-EVC. Vertical curves are measured as the horizontal distance between grade points, regardless of the slope of the grade. Table 79.3 lists the standard abbreviations used to describe geometric elements of vertical curves.

A vertical parabolic curve is completely specified by the two grades and the curve length. Alternatively, the *rate of grade change per station*, R, can be used in place of curve length. The rate of grade change per station is given by Eq. 79.46. Units of %/sta are the same as ft/sta.

$$R = \frac{G_2 - G_1}{L} \quad \text{[may be negative]} \qquad 79.46$$

Equation 79.47 defines an equal-tangent parabolic curve. (See Fig. 79.10.) x is the distance to any point on

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Table 79.4 *AASHTO Required Lengths of Curves on Grades*[a]

	stopping sight distance[b] (crest curves)	passing sight distance[c] (crest curves)	stopping sight distance (sag curves)
SI units			
$S < L$	$L = \dfrac{AS^2}{658}$	$L = \dfrac{AS^2}{864}$	$L = \dfrac{AS^2}{120 + 3.5S}$
$S > L$	$L = 2S - \dfrac{658}{A}$	$L = 2S - \dfrac{864}{A}$	$L = 2S - \dfrac{120 + 3.5S}{A}$
U.S. units			
$S < L$	$L = \dfrac{AS^2}{2158}$	$L = \dfrac{AS^2}{2800}$	$L = \dfrac{AS^2}{400 + 3.5S}$
$S > L$	$L = 2S - \dfrac{2158}{A}$	$L = 2S - \dfrac{2800}{A}$	$L = 2S - \dfrac{400 + 3.5S}{A}$

[a]$A = |G_2 - G_1|$, absolute value of the algebraic difference in grades, in percent.
[b]The driver's eye is 3.5 ft (1080 mm) above road surface, viewing an object 2.0 ft (600 mm) high.
[c]The driver's eye is 3.5 ft (1080 mm) above road surface, viewing an object 3.5 ft (1080 mm) high.

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The constants in Table 79.4 are based on specific heights of objects and driver's eyes above the road surface. In general, the sight distance over the crest of a vertical curve is given by Eq. 79.53 and Eq. 79.54, where h_1 is the height of the eyes of the driver and h_2 is the height of the object sighted, both in feet.

$$L = \frac{AS^2}{100\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2} \quad [S < L] \qquad \text{79.53}$$

$$L = 2S - \frac{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}{A} \quad [S > L] \qquad \text{79.54}$$

The sight distance under an overhead structure to see an object beyond a sag vertical curve is given by Eq. 79.55 and Eq. 79.56. C is the clearance between the road surface and the overhead structure. Various assumptions, including eye and taillight (object) heights and beam divergence, were made by AASHTO in developing the following equations.

$$L = \frac{AS^2}{800(C - 1.5)} \quad [S < L] \quad \text{[SI]} \qquad \text{79.55(a)}$$

$$L = \frac{AS^2}{800(C - 5)} \quad [S < L] \quad \text{[U.S.]} \qquad \text{79.55(b)}$$

$$L = 2S - \frac{800(C - 1.5)}{A} \quad [S > L] \quad \text{[SI]} \qquad \text{79.56(a)}$$

$$L = 2S - \frac{800(C - 5)}{A} \quad [S > L] \quad \text{[U.S.]} \qquad \text{79.56(b)}$$

21. DESIGN OF CREST CURVES USING *K*-VALUE

The *K-value method* of analysis used in the AASHTO *Green Book* is a simplified and more conservative method of choosing a stopping sight distance for a crest vertical curve. The length of vertical curve per percent grade difference, K, is the ratio of the curve length, L, to grade difference, A.

$$K = \frac{L}{A} = \frac{L}{|G_2 - G_1|} \quad \text{[always positive]} \qquad \text{79.57}$$

The $L = KA$ relationship is conveniently linear. In order to facilitate rapid calculation of curve lengths, AASHTO has prepared several graphs. Figure 79.12 and Fig. 79.13 give minimum curve lengths for crest vertical curves. Since for a fixed grade difference the speed determines the stopping distance, every value of speed has a corresponding value of K. Thus, the curves in the figures are identified concurrently with the speed and the K-value. It is not necessary to specify both A and L, as knowing K is sufficient.

The simplified procedure is to select one of the curves based on the speed or the K-value, and then read the curve length that corresponds to the grade difference, A. K-values shown on the graphs have been rounded for design.

The minimum curve lengths in the AASHTO figures have been determined by other overriding factors, the most important of which are experience and state requirements. Curve lengths calculated from Table 79.4 for $S > L$ often do not represent desirable design practice and are replaced by estimated values of three times the design speed (0.6 times the design speed in km/h). This is consistent with the minimum curve lengths of 100–300 ft (30–90 m) prescribed by most states. The estimated solutions in the AASHTO figures are also justified on the basis that the longer curve lengths are obtained inexpensively when the difference in grades is small.

Example 79.9

A car is traveling up a 1.25% grade of a crest curve with a design speed of 40 mph. The descending grade is −2.75%. What is the required length of curve for minimum proper stopping sight distance?

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Figure 79.12 *Design Controls for Crest Vertical Curves (SI units)*

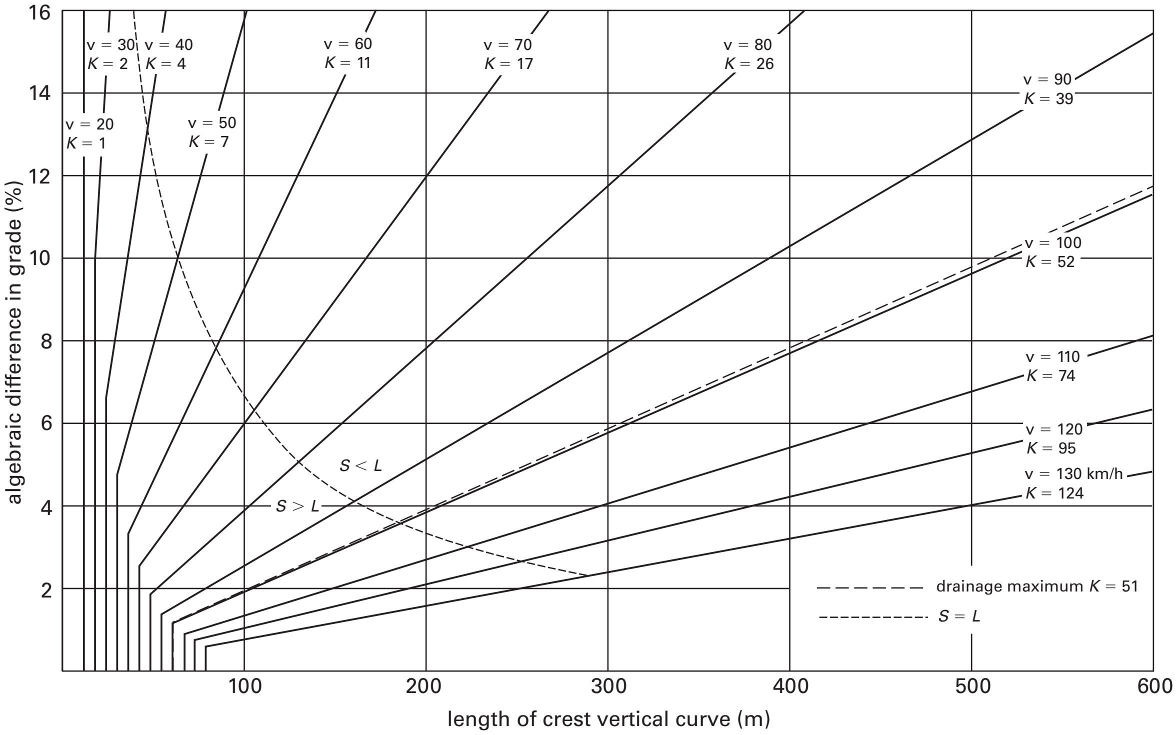

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Figure 79.13 *Design Controls for Crest Vertical Curves (customary U.S. units)*

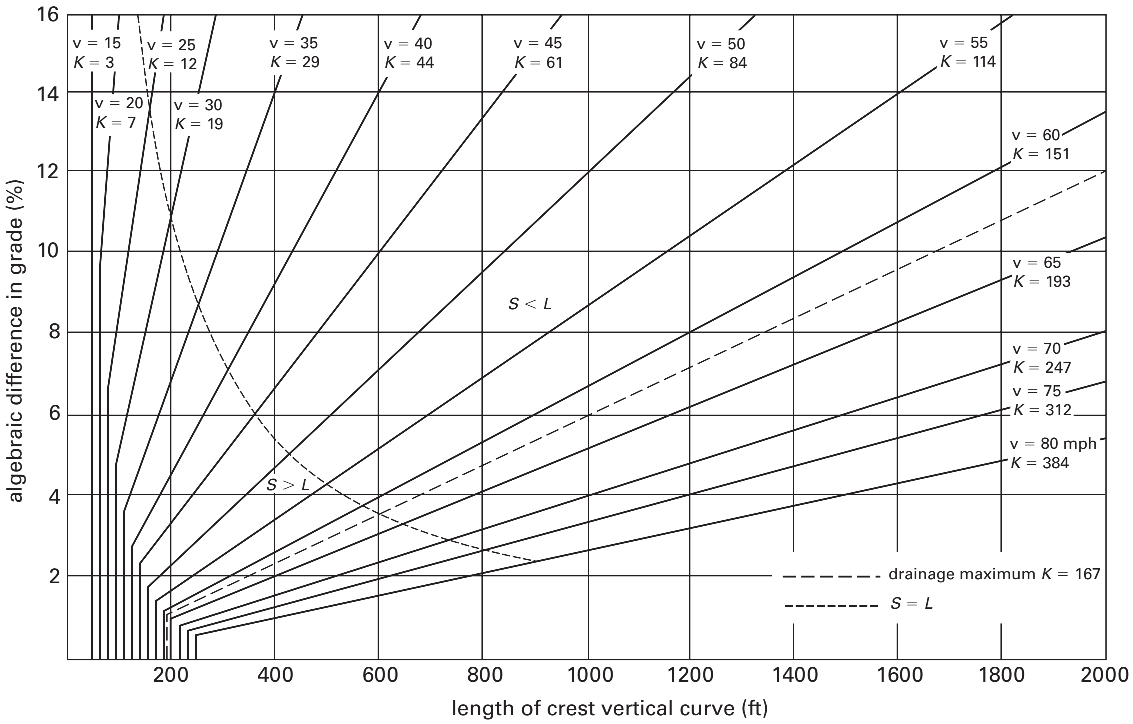

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Figure 79.14 *Design Controls for Sag Vertical Curves (SI units)*

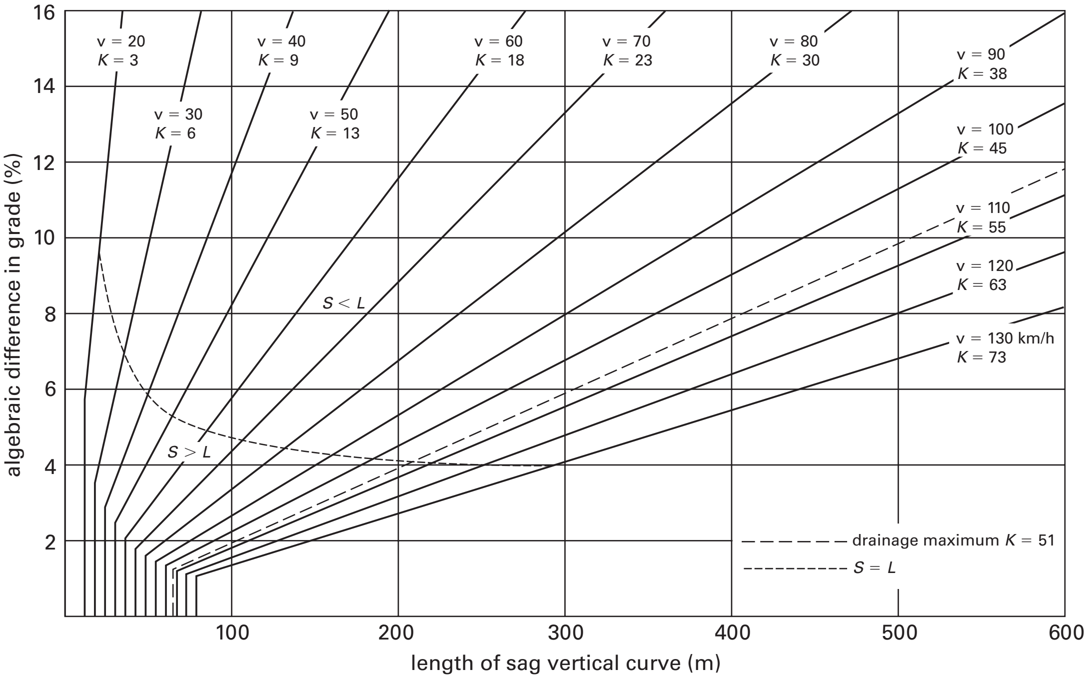

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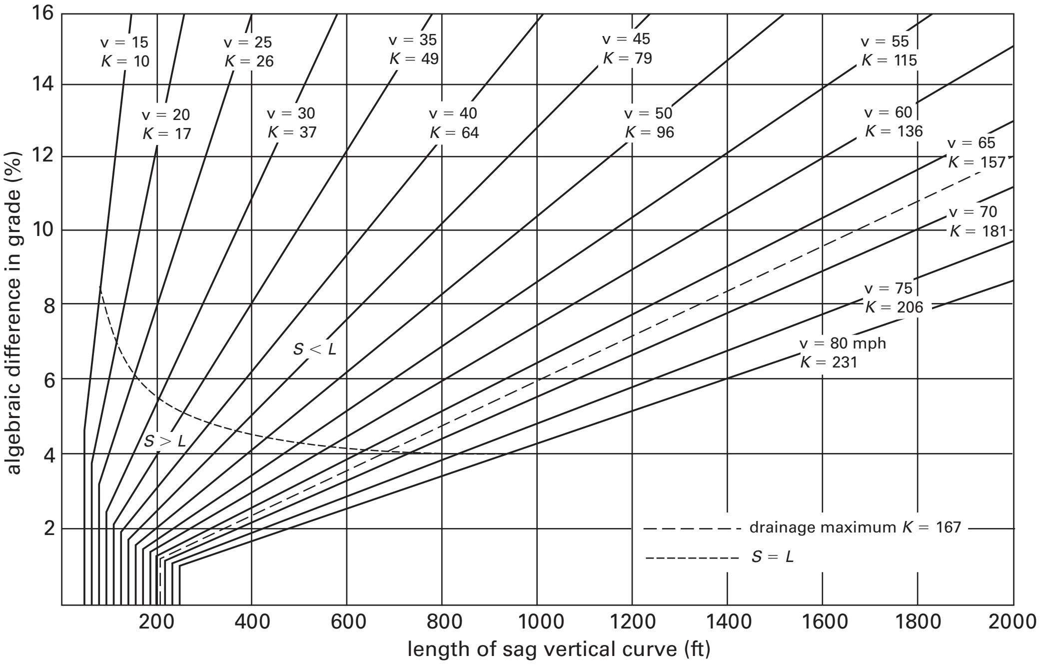
Figure 79.15 *Design Controls for Sag Vertical Curves (customary U.S. units)*



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Because $9° < 20°$, the tangent distance can be approximated as

$$\begin{aligned} x_c &= \frac{y_c}{\tan\frac{I_s}{3}} = \frac{15.71\ \text{ft}}{\tan\frac{9°}{3}} \\ &= 299.76\ \text{ft} \end{aligned}$$

From Fig. 79.17,

$$\begin{aligned} x_o &= x_c - R_c\sin I_s = 299.76\ \text{ft} - (954.93\ \text{ft})(\sin 9°) \\ &= 150.38\ \text{ft} \end{aligned}$$

Use Eq. 79.70.

$$\begin{aligned} Q &= \frac{L_s^2}{6R_c} - R_c(1-\cos I_s) \\ &= \frac{(300\ \text{ft})^2}{(6)(954.93\ \text{ft})} - (954.93\ \text{ft})(1-\cos 9°) \\ &= 3.95\ \text{ft} \end{aligned}$$

Use Eq. 79.69.

$$\begin{aligned} T_c &= (R_c + Q)\tan\frac{I}{2} \\ &= (954.93\ \text{ft} + 3.95\ \text{ft})\left(\tan\frac{85°}{2}\right) \\ &= 878.65\ \text{ft} \\ T_s &= x_o + T_c = 150.38\ \text{ft} + 878.65\ \text{ft} = 1029.03\ \text{ft} \end{aligned}$$

(a) Use Eq. 79.12.

$$\begin{aligned} \text{sta TS} &= \text{sta PI} - T_s = (\text{sta } 70{+}00) - 1029.03\ \text{ft} \\ &= \text{sta } 59{+}70.97 \end{aligned}$$

(b) Use Eq. 79.11.

$$\begin{aligned} \text{sta SC} &= \text{sta TS} + L_s = (\text{sta } 59{+}70.97) + 300\ \text{ft} \\ &= \text{sta } 62{+}70.97 \end{aligned}$$

(c) The station of CS is

$$\begin{aligned} \text{sta CS} &= \text{sta SC} + L_c = (\text{sta } 62{+}70.97) + 1116.67\ \text{ft} \\ &= \text{sta } 73{+}87.64 \end{aligned}$$

(d) The station of SI is

$$\begin{aligned} \text{sta ST} &= \text{sta CS} + L_s = (\text{sta } 73{+}87.64) + 300\ \text{ft} \\ &= \text{sta } 76{+}87.64 \end{aligned}$$

27. SPIRAL LENGTH TO PREVENT LANE ENCROACHMENT

Another consideration in determining spiral curve length is a driver's tendency to wander into an outer lane or onto the shoulder. A spiral curve that transitions too quickly may result in unsafe lane encroachment. Generally, the minimum curve length to prevent lane encroachment is less than the minimum curve lengths for comfort and superelevation. Nevertheless, the AASHTO *Green Book* provides two methods of calculating the minimum curve length based on encroachment prevention. The first method uses Eq. 79.62 with a C value 4.0 ft/sec^3 (1.2 m/s^3) [AASHTO *Green Book* Eq. 3-~~29~~].

The second method recognizes that most drivers are able to maintain their desired track only within 0.66 ft (0.20 m) of the desired direction in a spiral curve. This minimum shift (accuracy) is designated as the *minimum lateral offset between the tangent and the circular curve*, p_{min}. The AASHTO *Green Book* Eq. 3-~~28~~ uses this parameter and the radius of the adjacent circular curve to calculate the minimum spiral curve length.

$$L_{s,\text{min,m}} = \sqrt{24p_{\text{min,m}}R_{\text{m}}} \qquad \text{[SI]} \qquad 79.71(a)$$

$$L_{s,\text{min,ft}} = \sqrt{24p_{\text{min,ft}}R_{\text{ft}}} \qquad \text{[U.S.]} \qquad 79.71(b)$$

Similar reasoning limits the maximum length of a spiral curve. That is, if a spiral curve is too long, the driver may be mislead about the sharpness of the circular curve ahead. The maximum lateral shift that occurs with most drivers, p_{max}, is designated as the *maximum lateral offset between the tangent and the circular curve.* The standard value for p_{max} is 3.3 ft (1.0 m).

$$L_{s,\text{max,m}} = \sqrt{24p_{\text{max,m}}R_{\text{m}}} \qquad \text{[SI]} \qquad 79.72(a)$$

$$L_{s,\text{max,ft}} = \sqrt{24p_{\text{max,ft}}R_{\text{ft}}} \qquad \text{[U.S.]} \qquad 79.72(b)$$

28. MAXIMUM RADIUS FOR USE OF A SPIRAL

If a circular curve has a sufficiently large radius, a spiral curve may not be necessary. According to the AASHTO *Green Book*, only circular curves with radii less than a limiting maximum radius experience benefits of incorporating spiral curves. The limiting maximum radius will depend on design speed and is based on the desired maximum lateral (centripetal) acceleration, which varies between 1.3 ft/sec^2 and 4.25 ft/sec^2 (0.4 m/s^2 to 1.3 m/s^2), depending on the specifying agency. AASHTO

Green Book Table 3-~~20~~ lists the maximum radius based on a limit of 4.25 ft/sec^2 (1.3 m/s^2), as calculated from Eq. 79.73.

$$R_{\text{max,m}} = \frac{\text{v}^2}{a} = 0.0592\text{v}^2_{\text{km/h}} \quad \text{[SI]} \qquad 79.73(a)$$

$$R_{\text{max,ft}} = \frac{\text{v}^2}{a} = 0.506\text{v}^2_{\text{mph}} \quad \text{[U.S.]} \qquad 79.73(b)$$

29. DESIRABLE SPIRAL CURVE LENGTH

The *desirable spiral curve length* is simply the distance that most drivers would naturally take to transition to a circular curve. AASHTO *Green Book* Table 3-~~21~~ lists the desirable length of spiral curve transitions. The values are based on the distance traveled by the driver in two seconds at the roadway's design speed, which generally corresponds to the natural spiral paths taken by most drivers.

$$L_{s,\text{desirable,m}} = 0.556\text{v}_{\text{km/h}} \quad \text{[SI]} \qquad 79.74(a)$$

$$L_{s,\text{desirable,ft}} = 2.93\text{v}_{\text{mph}} \quad \text{[U.S.]} \qquad 79.74(b)$$

Deviations from the desirable spiral length are sometimes unavoidable. When shorter lengths are needed, they should never be less than $L_{s,\text{min}}$. Larger lengths (up to twice the desirable spiral length) may be needed in order to develop proper superelevation. Keep in mind that deviations from the desirable spiral length will result in drivers allowing their vehicles to shift laterally into adjacent lanes or shoulders. Such encroachment can be accommodated by increasing the lane widths in the spiral curve.

30. GEOMETRIC DESIGN OF AIRPORTS

The primary document governing geometric design of airports is FAA advisory circular AC 150/5300-13, *Airport Design*. Chapters cover airport layout, runway and taxiway design, taxiway bridges, line of sight, gradients, wind analysis, and other topics. Size and performance data are provided for typical aircraft.

Gradient limitations in AC 150/5300-13 are specified according to an *aircraft approach category* according to *expected landing speed*, $\text{v}_{\text{approach}}$. The expected landing speed is calculated as 1.3 times the stall speed in a landing configuration (i.e., at the certificated maximum flap setting with landing gear down) and maximum landing weight at standard atmospheric conditions. In keeping with world-wide convention, aircraft speeds are listed in *knots* (kt, nautical miles per hour), where 1 *nautical mile* (nm), is approximately 1.15078 statute miles.

category A: $\text{v}_{\text{approach}} < 91$ kt
category B: 91 kt $\leq \text{v}_{\text{approach}} < 121$ kt
category C: 121 kt $\leq \text{v}_{\text{approach}} < 141$ kt
category D: 141 kt $\leq \text{v}_{\text{approach}} < 166$ kt
category E: $\text{v}_{\text{approach}} \geq 166$ kt

Airports are often categorized as *utility airports* (roughly corresponding to airport approach categories A and B) and *transport airports* (roughly corresponding to airport approach categories C and D). Utility airports are designed to accommodate general aviation (i.e., small) aircraft and may be referred to as *general aviation airports*, while transport airports are designed to accommodate commercial passenger aircraft. Table 79.6 lists geometric characteristics of airport runways and taxiways.

31. GEOMETRIC DESIGN OF RAILWAYS

The geometric design of railways depends on the terrain, nature (freight, passenger, light rail, etc.), length, volume (number of vehicles per period), and speed of the rail traffic. The *Manual for Railway Engineering*, published by the American Railway Engineering and Maintenance-of-Way Association (AREMA), and the *Track Safety Standards Compliance Manual*, published by the U.S. Federal Railroad Administration (FRA), are standard resources. Although AREMA and FRA publications provide general guidance for alignment design, railways and transportation agencies usually have their own design standards. Accordingly, only the most general recommendations can be given. Table 79.7 lists general recommendations for maximum rates of grade change for vertical railroad curves.