Roadway Geometric Design I:
Functions, Controls and Alignments

Course No: C05-023
Credit: 5 PDH

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INTRODUCTION

Geometric design is the assembly of the fundamental three-dimensional features of the highway that are related to its operational quality and safety. Its basic objective is to provide a smooth-flowing, crash-free facility. Geometric roadway design consists of three main parts: **cross section** (lanes and shoulders, curbs, medians, roadside slopes and ditches, sidewalks); **horizontal alignment** (tangents and curves); and **vertical alignment** (grades and vertical curves). Combined, these elements provide a three-dimensional layout for a roadway.

This course is the **first** in a series of three volumes that summarizes and highlights the geometric design process for modern roads and highways. Subjects covered include: *highway functions (classification systems); design controls and criteria (design vehicles, highway capacity, traffic characteristics);* and *elements of design (sight distance, horizontal and vertical alignments).* The contents of this document are intended to serve as guidance and not as an absolute standard or rule.

When you complete this course, you should be familiar with the general design guidelines for roadways. The course objective is to give engineers and designers an in-depth look at the principles to be considered when selecting and designing roads.

The *American Association of State Highway and Transportation Officials (AASHTO)* publishes and approves information on geometric roadway design for use by individual state transportation agencies. The majority of today's geometric design research is sponsored and directed by AASHTO and the Federal Highway Administration (FHWA) through the National Cooperative Highway Research Program (NCHRP).

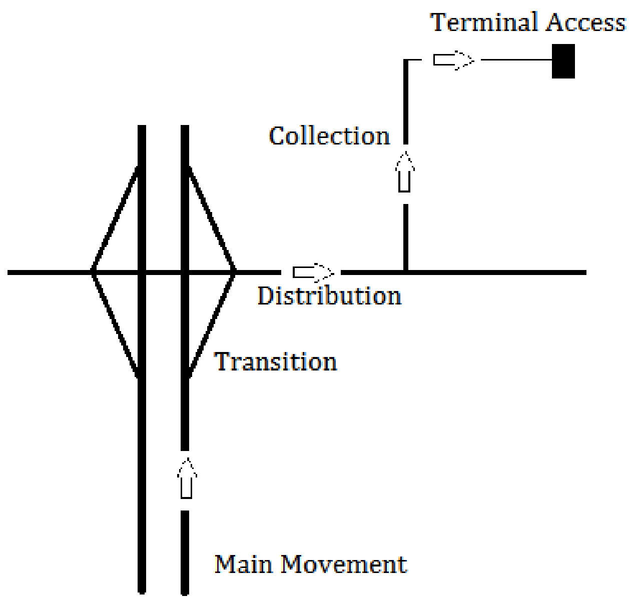
For this course, AASHTO's **A Policy on Geometric Design of Highways and Streets** (also known as the "Green Book") will be used primarily for fundamental geometric design principles. This text is considered to be the primary guidance for U.S. roadway geometric design.

This document is intended to explain some principles of good roadway design and show the potential trade-offs that the designer may have to face in a variety of situations, including cost of construction, maintenance requirements, compatibility with adjacent land uses, operational and safety impacts, environmental sensitivity, and compatibility with infrastructure needs.

HIGHWAY FUNCTIONS

FUNCTIONAL CLASSIFICATION

Functional classification refers to different types of roadways that are classified by the service they provide. Distinct travel movements of most trips include: *main movement, transition, distribution, collection, access, and termination.* The figure below illustrates a typical highway trip – where the freeway is the main movement (uninterrupted, high-speed), speed is reduced for the freeway ramps which act as transitions, arterials distribute vehicles near residential areas, collector roads enter the neighborhoods, and local access roads provide direst approaches to terminal facilities.



Hierarchy of Movement

Movement hierarchy is based on traffic volume with *freeway* travel typically at the highest level – *arterial distributors* are next with collectors and *local access roads* at the lowest levels. Roadway obsolescence occurs when designs fail to recognize and accommodate the different trip levels within the movement hierarchy. Inadequate functional transitions result in congestion and conflict between roadways and traffic generating facilities (i.e. commercial driveways, freeway ramps).

The functional classification of streets and highways is dependent on their character of service (nature and types of trips, basic purpose of the road, and general traffic volume). This type of classification recognizes that most traffic moves through road networks and can be classified by travel category relationships.

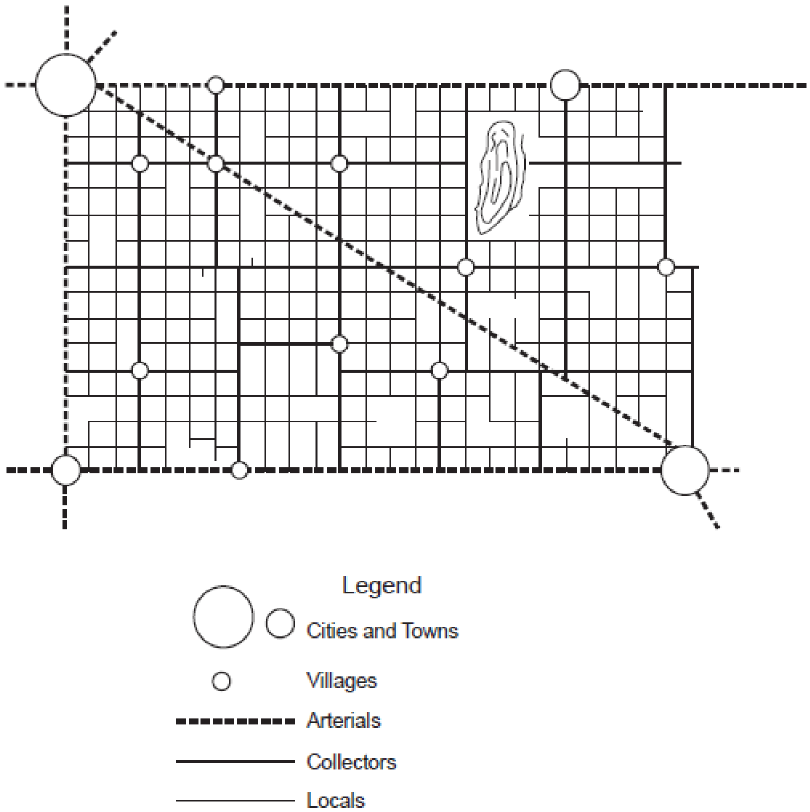


Figure 1-3. Schematic Illustration of a Functionally Classified Rural Highway Network

Access and mobility are crucial concerns for the functional classification of roadways. *Access-controlled roadways* (freeways, expressways, toll-ways) represent the top level of highway. Travel typically consists of longer distance trips with access limited to interchanges. These types of facilities utilize the most stringent design criteria to emphasize safety and mobility. *Arterial* roadways carry longer-distance traffic between cities and larger towns. They are usually designed with limits on driveways and interception spacing. *Collectors* connect and distribute traffic from local roads and small towns to the arterial networks. Although similar to local streets, collectors can handle substantially higher traffic volumes. *Local roads* serve as access to adjacent properties - intended for low traffic volumes at low speeds. Pedestrian safety is a high priority for these roadways.

FUNCTIONAL SYSTEM CHARACTERISTICS

Urban and rural roadway systems are classified separately due to their fundamentally different characteristics – *density, travel patterns, land use, and how these are related.* More arterials are generally used in urban areas with arterial subdivisions. Urban areas have set boundaries with a minimum population of 5000. Urbanized areas have populations of 50,000 or more while small urban area populations are between 5000 and 50,000.
Rural areas are those lands outside urban area boundaries. There are significantly more collectors and functional subdivisions of collector roads in rural areas.

Hierarchy of Functional Systems

Principal Arterials	*Main movement*
Minor Arterials	*Distributor*
Collectors	
Local Roads and Streets	

Rural Principal Arterial

- Suitable corridor movement for statewide and interstate travel
- Movements between urban areas with minimum 50,000 population and populations over 25,000
- Integrated movement without stub connections
- Interstate highways, freeways/expressways, and other principal arterials

Rural Minor Arterial

- Linkage of cities, larger towns, and other traffic generators
- Integrated interstate/inter-county systems
- Consistent internal spacing with population density
- Consistent corridor movements greater than rural collector or local systems
- High travel speeds and minimum interference to through movements

Rural collectors serve mainly intra-county traffic with shorter travel distances and more moderate speeds than arterials. These typically contain 20 to 25% of the total rural road length. *Major Collector Roads* serve important intra-county travel corridors, provides service to county seats larger towns (not on arterials), and links larger towns/cities with higher routes. *Minor Collector Roads* are reasonably spaced for population density to link important local traffic generators. They also provide service to smaller communities.

Rural local roads provide primarily short distance travel and land access adjacent to collectors. These roadways constitute all rural roads **not** in another classification and make up the majority (65 to 76%) of total rural roads.

Urban Principal Arterial

- For major urban areas, high traffic volumes, and largest trips
- Integrated internally and between major rural connections
- Carries important intra-urban and intercity bus routes
- Fully or partially access-controlled
- Spacing varies from 1 mile (central business districts) to 5 miles (less developed fringe areas)

Urban Minor Arterial

- For smaller areas, moderate length trips, and lower travel mobility
- Includes all arterials not classified as principal
- Places more emphasis on land access
- Does not typically penetrate neighborhoods
- Spacing ranges from 0.1 to 0.5 (central business areas) to 2 to 3 miles (suburban fringes) – normally 1 mile maximum in fully developed districts

Urban collectors provide service within residential, commercial, and industrial areas. These roadways penetrate neighborhoods and channel traffic to their ultimate destinations. Urban collectors may include total street grids in central business areas and carry local bus routes.

Urban local street systems contain all other roads not in a higher classification – 65 to 80% of total urban road length. These provide direct access to properties and connect to other facilities (arterials, collectors). Urban streets have the lowest mobility levels, no bus routes (if possible), and minimal service to through traffic.

ROADWAY DESIGN CONTROLS

DESIGN VEHICLES

The range of design vehicles and their physical/operating characteristics should be appropriate for the roadway design. These vehicles have critical dimensions and characteristics that influence and control the design of highway elements (widths, vertical clearances, corner radii, climbing lanes, etc.). General classes of design vehicles include: *passenger cars; buses; trucks;* and *recreational vehicles.* Please refer to AASHTO Table 2-1b for various design vehicle dimensions.

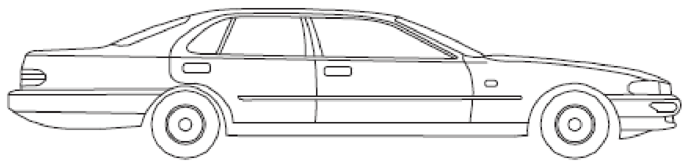
Design vehicle choices should be based on the roadway's functional classification and the types of vehicles expected. In urban areas, it is crucial to select a design vehicle that meets the site conditions.

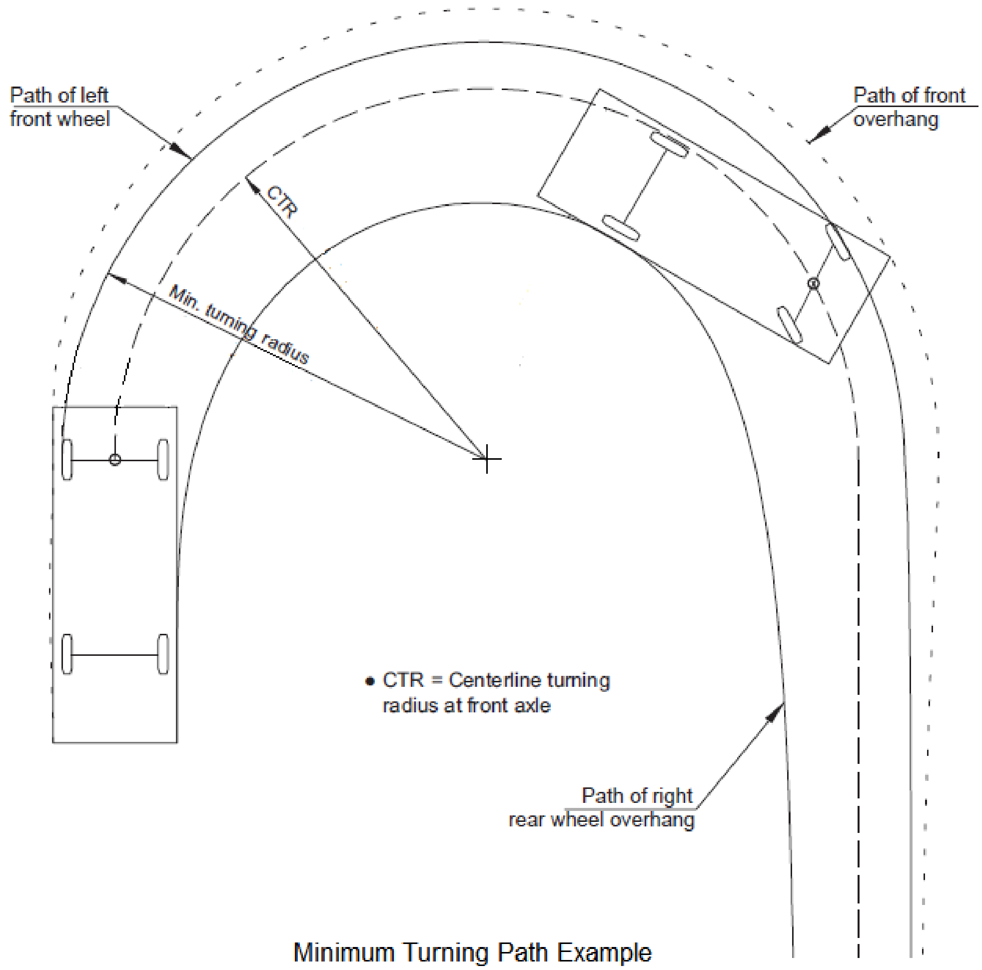
- A **passenger car (P)** may be used for parking lots
- A **city transit bus (CITY-BUS)** may be used for highway intersections with city streets designated as bus routes with low truck traffic
- A **two-axle single-unit truck (SU-30)** may be used for intersections of residential streets and park roads
- A **three-axle single-unit truck (SU-40)** may be used for collector streets and locations with larger single-unit truck traffic

Large (84-passenger) or conventional (65-passenger) school buses may be used as the design vehicle for highway intersections with low-volume county highways and roads under 400 ADT. An Interstate Semitrailer (WB-67) is the typical minimum size design vehicle used for freeway ramp/arterial intersections, high traffic highway/industrialized street intersections, or large truck access roads.

AASHTO design vehicles represent the critical dimensions for typical vehicle types found on the road. The turning characteristics of design vehicles show their turning paths for roadway designs. The primary turning path dimensions that impact roadway design are: *minimum centerline turning radius (CTR); out-to-out track width; wheelbase; and path of inner rear tire.*

Vehicle speeds for minimum turning radii are assumed to be less than 10 mph to minimize the effects of driver characteristics and wheel slip angles. Higher speeds require longer transitions and larger curves. Turning path boundaries for the sharpest turns are set by the outer trace of the front overhang and the inner rear wheel path. The sharpest turn of the outer front wheel is used to determine the minimum centerline turning radius. Trucks and buses require more area due to their widths, wheelbases, and greater minimum turning radii.





Minimum Turning Path Example

DRIVER PERFORMANCE

Geometric roadway design should always strive to meet driver needs. Highway designs that meet user capabilities and limitations aid their performance – while incompatible designs may increase driver errors, crashes, and inefficient operation.

The aging driving population and their influence on roadway safety continue to be a legitimate concern. The percentage of driving Americans 65 years of age and older is expected to increase to 22% by 2030. Some operational deficiencies of an aging population may include: *slower reaction times; hearing deterioration; limited depth perception; visual deterioration; slower decision making; limited physical mobility; slower information processing; decline in judgment ability;* and *drug side effects.* To help combat these conditions, design criteria for intersections, sight distance, and signing have been improved to meet the needs of older drivers. Countermeasures for older drivers include:

- Designing for 95th or 99th percentile driver (when practical)
- Improving sight distances – stopping, intersection, decision
- Evaluating sight triangle adequacy
- Simplifying intersections/interchanges with multiple information processing
- Increasing protected left-turn signals
- Considering alternate designs with minimal conflicts
- Increasing vehicle clearance signal times
- Using wider, brighter pavement markings and signs
- Enforcing roadway speed limits

The most important roadway design accommodation for older drivers is increasing sight distance → particularly **decision sight distance.**

TRAFFIC CHARACTERISTICS

Any roadway geometric design should jointly consider traffic volumes and distribution characteristics. Designers are typically interested in forecasts for a designated design year. These factors are based on the type of facility improvement and can greatly impact alignments, grades, number of lanes, and widths.

The **Average Daily Traffic (ADT)** is considered to be the most basic measure of roadway traffic volume and is defined as "the total volume during a given time period, greater than one day and less than one year, divided by the number of days in that time period".

$$\text{ADT} = \frac{\text{Total traffic volume during a time period}}{\text{Number of days in time period}}$$

Using ADT volumes may be more appropriate for the geometric design of low-volume local and collector roads (not highways) since it does not indicate traffic volume variations by time. Roadways designed for average traffic might need to carry volumes on some days that are higher than the ADT.

Shorter time intervals (usually one hour) for traffic volumes typically reflect more realistic operating conditions that can be used for roadway design. Traffic volumes vary depending on the time of day and time of year. The key is to determine which hourly volume should be used as a design control – this value should not be exceeded often or by very much. AASHTO recommends using the 30th highest hourly volume of the year (30 HV) for design purposes. The 30 HV should be determined from the 30th hour factors for similar roadways in the same location operating under similar conditions.

The **Design Hourly Volume (DHV)** for rural roadways is usually the 30 HV for the future design year. While the 30 HV criteria generally applies to urban areas, an appropriate DHV value may be gathered from traffic data during peak time periods.

Hourly traffic volumes for each direction of travel are crucial for highway design – especially locations with intersections or auxiliary lanes. Roadways with high percentages of peak hour traffic in one direction may require more lanes than a similar roadway with the same ADT but less directional traffic. The **Directional Factor (D)** or "D-factor" is a measurement of the dominant traffic flow direction. These peak values can range from 55 to 70 percent of the traffic for rural highways (occasionally reaching 80%). Current directional distributions can be applied to forecast future year DHV values.

Directional Design Hour Volume (DDHV) is the directional distribution of multilane traffic during the design hour. DDHV for multilane roadways is the product of multiplying the ADT by the 30 HV percentage of the ADT by the percentage of peak direction traffic by the design hour.

Example: DHV = 25% of ADT
Directional Distribution (peak hour) = 55:45
DDHV = 0.25 x 0.55 x ADT → 13.75% of the ADT

The DDHV can also be determined by using the Directional Factor (D)

DDHV = D x DHV

In cases where only one direction of the ADT is known – the ADT is approximately twice the directional ADT value.

Roadway design should consider vehicles of different sizes, weights, and operating characteristics. These are generally grouped into the following classes:

- *Passenger cars (cars, vans, pickup trucks, and SUVs)*
- *Trucks (buses, single-unit trucks, combination trucks, and RVs)*

Trucks are defined as having a manufacturer's gross vehicle weight rating of 9000 lbs or more with dual tires on a minimum of one rear axle. These vehicles have a greater impact on traffic operations versus passenger cars – high percentages of trucks produce greater equivalent traffic demand and require greater highway capacity. Traffic studies should provide traffic composition data for roadway projects. The percentage of peak hour truck traffic is needed for design purposes. The average percentages of truck traffic for a number of weekly peak hours are suitable for roadway design.

Geometric designs for roadways should be based on future traffic volumes expected for the facility. Roadway life expectancies may vary due to different factors – *obsolescence, land use, traffic volumes, vehicle patterns, and demands.*

	Physical Life Expectancy
Right-of-way and grading	100 years
Minor drainage structures & base courses	50 years
Bridges	25 to 100 years
Resurfacing	10 years
Pavement structure	20 to 30 years

Design volumes should be able to be estimated with reasonable precision. The maximum design period is commonly accepted as 15 to 24 years with a period of **20 years** widely used for roadway design purposes.

SPEED

Speed is a crucial factor for selecting alternative routes or transportation modes.
Roadway designs should integrate the concept of speed consistency into their plans to produce modest speed changes between road segments. Vehicle speeds depend on: *physical characteristics; roadside interferences; weather; vehicle presence; and speed limitations.*

Operating Speed is the observed vehicle speed during free-flow conditions – the 85th percentile is typically the operating speed used.

Running Speed is the speed of travel for vehicles over a roadway section.

$$\text{Running Speed} = \frac{\text{Length of Roadway Section}}{\text{Vehicle Running Time}}$$

Average Running Speed is the most appropriate speed for evaluating levels of service (LOS) and user costs. These vary during the day and depend on traffic volume. When used, running speed should be stated as peak hour, off-peak hour, or daily average. Peak and off-peak values are suitable for design and operation while average speeds for entire days can be used economically.

AASHTO defines **Design Speed** as "the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern". Design speed is a primary factor in roadway design which may equal or exceed the legal statutory speed limit. It is the selected speed for determining various roadway geometric designs. Design speed is used as an overall design control for horizontal alignments in roadway design. The level of service is directly related to the facility's speed of operation. It should be compatible with driver expectation and consistent with the roadway's functional classification and location. Other characteristics not directly related to design speed may affect vehicle speeds. Therefore, changes to design speed may result in changes to many elements of the roadway design.

Posted speed limits are typically the 85th percentile speed of traffic – within a 10 mph speed range of most drivers. There are important differences between low- and high-speed design criteria which produce a 45 mph upper limit for low-speed designs and a 50 mph lower limit for high-speed designs.

HIGHWAY CAPACITY

AASHTO defines capacity as "the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point (i.e., a uniform section of a lane or a roadway) during a given time period under prevailing roadway and traffic conditions". Roadways are usually designed to operate at traffic volumes less than their capacity to avoid congestion and breakdown flow.

The Highway Capacity Manual (HCM) is commonly used to determine roadway capacities. These capacity analyses are useful for the following: **Highway Design** *(matches proposed roadways to traffic demand and selects roadway types and dimensions);* **Transportation Planning Studies** *(evaluates current service levels for existing traffic and estimates when future traffic may exceed capacity);* and **Traffic Operational Analyses** *(identifies bottleneck locations, and estimates resulting operational improvements from proposed measures).*

Road designers use traffic volumes to determine the number of travel lanes, special requirement needs, or design details (lengths, tapers, channelization, etc.). The primary goal is to produce a road with suitable dimensions and an adequate alignment to support the design service flow rate (minimum value equal to flow rate during peak 15-minute period of the design hour).

> **Design Volume** is the projected traffic volume to use for the design year (typically 10 to 20 years in the future) – often stated as an expected traffic volume for a specific design hour.

Design Service Flow Rate is the maximum hourly traffic flow rate that a particular road can handle without an unacceptable degree of congestion.

Key considerations of geometric design include roadway design, roadway traffic, and degrees of congestion. Unlike the first two key considerations, exact units for the degrees of congestion are sometimes difficult to express – crash frequencies, severity, maneuverability, traffic volume to capacity ratio (v/c), operating speed, and average running speed have all been suggested.

For uninterrupted traffic flow – *speed, density, and volume/rate of flow* are measures of operational conditions.

For interrupted flow – *average stopped-time delay* is the main measure of effectiveness.

Roadway Congestion is a restriction or interference to normal free flow traffic. As flow rates increase, congestion increases until it meets capacity and becomes a serious problem (stop-and-go traffic, reduced travel speeds, higher travel times, traffic backups, etc.).
The degree of congestion is directly related to the resources available. The desires of the road user should be reconciled with the available resources for satisfaction. Degrees of congestion can be assessed by the following steps: *determine satisfactory operating conditions for the majority of drivers; determine the most extensive (but practical) roadway improvement allowed by governmental authorities; and reconcile public demands with available finances.*

The **Peak Hour Factor (PHF)** converts traffic flow rates for the most congested 15-minute span to the total hourly volume. Operating conditions during the highest 15-minutes determine the hourly service level.

$$\mathrm{PHF} = \frac{\text{Total Hourly Volume}}{4 \text{ x (No. of vehicles during the highest 15 minute period)}}$$

or

$$\mathrm{PHF} = \frac{\text{Actual Hourly Volume}}{\text{Peak Rate of Flow}}$$

Peak hour factors typically fall within the 0.75 to 0.95 range – never exceeding 1.00.

Levels of Service (LOS) are used to classify users' quality of traffic flow on a specific facility. These levels range from **A** (free flow) to **F** (forced or breakdown flow) and uses traffic performance measures (speed, travel time, maneuverability, traffic, driver comfort, and convenience) for evaluation. Design LOS depends on the specific facility type and terrain. Low LOS for design year traffic may produce lower flow qualities during peak periods – while high LOS values may result in expensive and unacceptable designs. Please refer to the *Highway Capacity Manual (HCM)* for further information regarding levels of service.

Table 2-5. Guidelines for Selection of Design Levels of Service

Functional Class	Appropriate Level of Service for Specified Combinations of Area and Terrain Type			
	Rural Level	Rural Rolling	Rural Mountainous	Urban and Suburban
Freeway	B	B	C	C or D
Arterial	B	B	C	C or D
Collector	C	C	D	D
Local	D	D	D	D

ACCESS CONTROL

Access Control is the regulation of access to abutting lands in connection with roadways controlled by public authorities – resulting in preservation of service, crash reduction, and management of through traffic interference. Managed access provides entrances and exits that best fit traffic and land usage with minimal through traffic conflicts. Categories of access control include:

Full Control of Access

Preference is given to through traffic; Access provided via ramps to select public roads; At-grade crossings and direct driveways are prohibited

Partial Control of Access

Some preference to through traffic; At-grade or grade-separated ramps to select roads and driveways

Access Management

Provides land access while preserving traffic flow; Sets access policies and designs to be incorporated into legislation; Views roadways and surrounding areas as a single system (holistic)

Driveway/Entrance Regulations

Sets limits for access location, number, and geometric design; Provides access for each abutting property

AASHTO's basic principles that define access management include: *classifying the road system by each roadway's primary function; limiting access to higher functional roads; locating traffic signals to emphasize traffic movements; using curbed medians and median openings to manage access: and locating driveways/entrances to minimize traffic interference.* Transportation agencies can control access through the use of design policies, driveway regulations, legal statutes, and land-use ordinances. Good access management can help limit the increase in crashes and travel times associated with increased access density.

PEDESTRIAN FACILITIES

Due to roadway interactions between pedestrians and motorized traffic, it is critical to integrate these during the project planning and design phases. The *Americans with Disabilities Act (ADA) of 1990* also requires that any new or reconstructed pedestrian facilities (sidewalks, shared-use paths, shared streets, or off-road paths) **must** be accessible to disabled individuals.

Typical pedestrians will not walk over 1 mile to work or over 0.5 mile to catch public transportation. Approximately 80% of distances traveled will be less than 0.5 mile. Pedestrians shop 50% of the time and commute only 11 percent.

The typical range of values for walking speed varies from 2.5 to 6.0 ft/sec^2. The MUTCD recommends using **4.0 ft/sec^2** as the walking speed value when calculating pedestrian clearance intervals for signalized intersections. Advanced age is the common cause of slower walking speeds → 2.8 ft/sec^2 is an appropriate design speed for older pedestrian areas.

Measures for reducing pedestrian-vehicular conflicts include: *eliminating left/right turns; prohibiting free-flow right turn movements; eliminating right turn on red; converting two-way streets to one-way operations; providing separate pedestrian signal phases; eliminating selected sidewalks;* and *providing pedestrian grade separations.* In order to accommodate pedestrians with visual, hearing, or cognitive impairments, various types of information (auditory, tactile, and kinesthetic) should be combined to render assistance. Different treatments may include: curb ramps; pedestrian islands; fixed lighting; pedestrian signals; audible signals; etc.

BICYCLE FACILITIES

Bicycles continue to be a popular mode of transportation and their facilities should be a major consideration for any roadway design. The main factors to consider for accommodating bicycles include: *type of bicyclist being served by the route (experienced, novice, children); type of roadway project (widening, new construction, resurfacing); and traffic operations & design characteristics (traffic volume, sight distance, development).*

Existing roads and streets provide the majority of the required network for bicycle travel. Designated bikeways may be needed at certain locations to supplement the existing road system. Transportation planners and designers list the following factors as greatly impacting bicycle lanes: *traffic volume; average operating speed; traffic mix; on-street parking; sight distance; and number of intersections.*

Basic Types of Bicycle Facilities

Shared lane: typical travel lane shared by both bicycles & vehicles
Wide outside lane: outside travel lane (14 ft min.) for both bicycles & vehicles
Bicycle lane: part of roadway exclusively designated for bicycles, etc.
Shoulder: roadway paving to the right of traveled way for usage
Multiuse path: physically separated facility for bicycles, etc.

At locations without bicycle facilities, other steps need to be considered for enhancing bicycle travel on roads and streets. Low to moderate cost improvements that can help to reduce crash frequency and allow for bike traffic include: *paved roadway shoulders; wider outside traffic lanes (14 ft min.) – if no shoulders; bicycle-friendly drainage grates; manhole covers at grade; and smooth, clean riding surface.*

AASHTO's *Guide for Development of Bicycle Facilities* provides specific guidance regarding bicycle dimensions, operation, and needs – which determine acceptable turning radii, grades, and sight distance. The main differentiation between bikes and other vehicles is that the bicycle and rider are considered together as a system. Driver characteristics for motor vehicles are important but the driver-vehicle interface is rarely considered.

Typical bicyclist requirements: 3 feet lateral space 7.5 feet height

Required track width: 0.7 feet @ 7 mph or greater 2.5 feet @ 3 mph or greater

These track widths are not comfortable for riders – greater separation from traffic and more maneuvering space is preferable.

ENVIRONMENT

Roadways should be designed to complement their surroundings (social, physical, natural, etc.) and be considered a component of the total interrelated system. The environment plays a crucial role in the use of certain roadway geometric design features. Topography (level, rolling, or mountainous terrain) significantly influences the alignment, road cross section, traffic operations, and project cost-effectiveness. Changes in one part of the system tend to affect other variables – these may be negligible or serious. Climate can also play a significant role in roadway design. Road drainage affects cross-sectional features (e.g., cross slope, superelevation), alignment controls (e.g., minimum grades), and snow/ice control (shoulders, use of raised channelization, and cross slope).

DESIGN ELEMENTS

SIGHT DISTANCE

Sight distance is essential for the safe and efficient operation of vehicles. It is the length of roadway ahead where an object of specified height is continuously visible to a driver. This distance is dependent on the driver's eye height, the specified object height, and the height/position of sight obstructions.

Sight Distance Criteria

Height of Driver's Eye:	3.50 feet above road surface (passenger vehicles)
	7.60 feet above road surface (trucks)
Height of Object:	2.00 feet above road surface (stopping & decision)
	3.50 feet above road surface (passing & intersection)

Stopping Sight Distance

Stopping sight distance is considered to be the most basic form of sight distance. It is defined as the length of roadway needed for a vehicle traveling at design speed to stop before colliding with an object in its path. Ideally, all of the roadway should provide stopping sight distance consistent with its design speed. However, this distance can be affected by both horizontal and vertical geometric features.

Stopping sight distance consists of two distances:

Brake Reaction Distance – Length of roadway travelled by the vehicle from driver perception to brake application. A brake reaction time of 2.5 seconds is the recommended design value and exceeds the 90th percentile for all drivers. This criterion has proven to be inadequate for most complex driving conditions encountered on the roadway.

Braking Distance – Roadway distance required to stop the vehicle from the instant of brake application. It may be calculated from the following equation (Equation 3-1):

$$d_B = 1.075 \frac{V^2}{a}$$

where:

d_B = braking distance, ft

V = design speed, mph

a = deceleration rate, ft/s^2

A recommended rate for deceleration is **11.2 ft/s²**.

If the roadway is on a grade, this distance can be determined by Equation 3-3:

$$d_B = \frac{V^2}{30\left[\left(\frac{a}{32.2}\right) \pm G\right]}$$

where:

d_B = braking distance on grade, ft

V = design speed, mph

a = deceleration, ft/s^2

G = grade, rise/run, ft/ft

Stopping sight distance is a function of initial speed, braking friction, perception/reaction time, and roadway grade. It also contains assumptions about the driver's eye height (3.5 feet) and the size of object in the road (2 feet above the road surface). This distance can be determined from the following equation (Equation 3-2):

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

where:

SSD = stopping sight distance, ft

V = design speed, mph

t = brake reaction time, 2.5 s

a = deceleration rate, ft/s^2

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		Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance on Level (ft)	Stopping Sight Distance	
			Calculat-ed (m)	Design (m)				Calculat-ed (ft)	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/s^2 [11.2 ft/s^2] used to determine calculated sight distance.

Limitations of the AASHTO model

- Not fully accounting for heavy vehicles (longer stopping times/distances)
- Not differentiating for various highway types
- Not recognizing differing conditions along the same highway.

Decision Sight Distance

Design policy recognizes that unexpected conflicts, driver decisions, or changes in the roadway can require additional time, and longer distances for the motorist to make a decision and safely navigate the vehicle. This sight distance provides greater visibility for drivers and is referred to as **decision sight distance**. It is the length of roadway required to detect a potential threat/condition, select an appropriate reaction, and perform the resulting maneuver. This distance also provides additional margin for error for vehicle maneuverability. Decision sight distance should be provided for interchanges, intersections, lane drops, alignment changes, bridges, changes in roadway cross-section, and toll collection facilities.

Decision sight values depend on the roadway's location (rural or urban) and the type of avoidance maneuver.

Avoidance Maneuver	**Condition**	**Time (sec)**
A	Stop on rural road	3.0
B	Stop on urban road	9.1
C	Change on rural road	10.2 to 11.2
D	Change on suburban road	12.1 to 12.9
E	Change on urban road	14.0 to 14.5

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	235	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 distances for **Avoidance Maneuvers A and B** can be calculated using the following formula (Equation 3-4):

$$DSD = 1.47Vt + 1.075\frac{V^2}{a}$$

where:

DSD = decision sight distance, ft

t = pre-maneuver time, s (see notes in Table 3-3)

V = design speed, mph

a = driver deceleration, ft/s^2

Decision sight distances for **Avoidance Maneuvers C, D, and E** can be calculated from the following equation (Equation 3-5):

$$DSD = 1.47Vt$$

where:

DSD = decision sight distance, ft

t = total pre-maneuver and maneuver time, s (see notes in Table 3-3)

V = design speed, mph

<u>Passing Sight Distance</u>

Passing sight distance is the length of roadway available to drivers on two-lane two-way highways to pass slower vehicles. Minimum values for passing sight distances are based on warrants for no-passing zones on two-lane highways as presented in the **Manual on Uniform Traffic Control Devices (MUTCD).**

Potential conflicts in passing operations are ultimately determined by driver response to: *view of roadway ahead provided by available PSD*; and *passing and no-passing markings.*

AASHTO Table 3-4 shows design values for passing sight distance on two-lane highways. Research has shown that more sight distance is needed for passing maneuvers than for stopping sight distance which is continuously provided along roadways.

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

Assumptions Regarding Driver Behavior

- Passing and opposing vehicle speeds are equal to the roadway design speed
- Speed differential between passing and passed vehicle is 12 mph
- Passing vehicle has adequate acceleration capability to reach speed differential (40% of way through passing maneuver)
- Vehicle lengths are 19 feet
- Passing driver's perception-reaction time to abort passing maneuver is 1 second
- Deceleration rate of 11.2 ft/s^2 for passing vehicle when passing maneuver is aborted
- Space headway between passing and passed vehicles is 1 second
- Minimum clearance between passing and opposed vehicles upon return to normal lane is 1 second

Passing sight distances should be equal or greater than the minimum values. Also, they should be as long and frequent as possible, depending on: **topography; design speed; cost; and intersection spacing**

Intersection Sight Distance

Intersection sight distance is the length of roadway along the intersecting road that the driver on the approach should have to perceive and react to potential conflicts. Approaching drivers should have an unobstructed view of the intersection and adequate roadway length to anticipate/avoid potential crashes. Intersection sight distances that exceed stopping sight distances are desirable along major roads for enhancing traffic operations. Methods for determining intersection sight distances are based on the same principles as stopping sight distance.

Sight triangles are areas along intersection approach legs that should be clear of obstructions that could block a driver's view of potential conflicts. The dimensions are based on driver behavior and depend on roadway design speeds and type of traffic control. Object height is based on a vehicle height of **4.35 feet**.

Recommended sight triangle dimensions vary for the following different types of traffic control:

- **Case A:** Intersections with no control
- **Case B:** Intersections with stop control on the minor road
  - **Case B1:** Left turn from the minor road
  - **Case B2:** Right turn from the minor road
  - **Case B3:** Crossing maneuver from the minor road
- **Case C:** Intersections with yield control on the minor road
  - **Case C1:** Crossing maneuver from the minor road
  - **Case C2:** Left or right turn from the minor road
- **Case D:** Intersections with traffic signal control
- **Case E:** Intersections with all-way stop control
- **Case F:** Left turns from the major road

HORIZONTAL ALIGNMENT

The **alignment** is the route of the road, defined geometrically as a series of horizontal tangents (straight roadway sections), circular curves, and spiral transitions. It depicts its three-dimensional location in relation to the terrain and adjacent land use. The primary objective of geometric roadway design is to assemble these elements in order to provide a compatible speed with the road's function and location. By establishing both horizontal and vertical roadway alignments, designers significantly influence safety and operational quality, in addition to construction and maintenance costs.

Rural Design Speeds

The design speed for a rural highway should be consistent with the terrain, roadway type, and location. For most rural highways, it should be as high as practicable to provide the greatest degree of safety and operational efficiency for the resulting design dimensions.

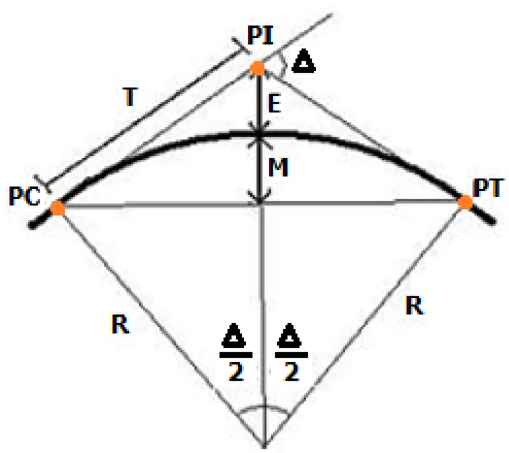
Studies have shown that motorists tend to operate comfortably at speeds that are higher than typical design speeds.

Urban Design Speeds

High speed designs may be inappropriate at certain locations (example: residential streets). Traffic calming techniques in sensitive areas have proven to be a viable option for traffic operations in residential areas. For urban arterials, designers should evaluate high speed compatibility with safety, pedestrians, driveway activity, parking, and other factors. Geometric roadway design should consider the location of horizontal curves; their "sharpness"; their tangent lengths; and their relationship to the vertical alignment.

HORIZONTAL CURVES

Horizontal curve design is based on physics and predicted driver response to lateral acceleration. The elements of curve design include: *curve radius; superelevation; side friction;* and *assumed vehicle speed.* Horizontal curve design requires the establishment of a minimum radius (based on speed limit), curve length, and objects obstructing the driver's view (sight distance). High speed horizontal curves with small radii need an increased superelevation (bank) for safety. For limited sight distance around a corner or curve, the designer must ensure drivers can see in order to avoid accidents.



Terms

R = Radius
PC = Point of Curvature (point at which the curve begins)
PT = Point of Tangent (point at which the curve ends)
PI = Point of Intersection (point at which the two tangents intersect)
T = Tangent Length
C = Long Chord Length (straight line between PC and PT)
L = Curve Length

M = Middle Ordinate or HSO – Horizontal Sightline Offset
(distance from sight- obstructing object to the middle of the outside lane)
E = External Distance
Δ= Deflection Angle

Side Friction Factor

The side friction factor depicts the lateral acceleration that acts on a vehicle, and shows the need for side friction. Varied speeds on horizontal curves produce tire side thrust which is counterbalanced by friction between the vehicle tires and the riding surface. The following basic side friction equation, Equation 3-7 (simplified curve formula) produces slightly higher friction estimates than those resulting from the "basic curve formula".

$$f = \frac{V^2}{15R} - 0.01e$$

Upper values for the side friction factor produce the point of impending skid – where the vehicle tire would begin to skid. Since highway curves are designed with a margin of safety to prevent skidding, the friction values used for design should be considerably less than those values at impending skid. The recommended side friction factors for rural highways, urban freeways, and high-speed urban roadways are shown in AASHTO Figure 3-6. Maximum values range from 0.14 at 50 mph to 0.08 at 80 mph. The key to selecting maximum side friction factors is the level of lateral acceleration that causes driver discomfort and reaction to avoid higher speeds.

Normal Cross Slope

Drainage needs of a roadway determine the minimum rate of cross slope applicable to the traveled way. Depending on the roadway type and weather conditions, acceptable minimum cross slope values range from 1.5 to 2.0 percent (with 2.0 representative for paved, uncurbed pavements).

Maximum Superelevation Rates

No single maximum superelevation rate is universally applicable due to: *climate (amount of precipitation); terrain (flat, rolling, or mountainous); area type (rural or urban); or slow-moving vehicles (frequency).* The upper limits for superelevation on a horizontal curve consider constructability, land usage, slow-moving vehicles, and climate. At locations with regular snow or ice, the superelevation rate should not exceed the rate where slow-moving vehicles would slide toward the center of the curve when icy.

Several maximum superelevation rates establish design controls for roadway curves. The highest superelevation rate for highways is typically 10 percent, with rates over 12%

considered impractical. For urban areas with few constraining factors, a lower maximum rate of superelevation (4 to 6 percent) may be applicable. On low-speed urban streets with severe constraints, a low maximum rate or no superelevation may be used.

Minimum Curvature

The minimum radius is a limiting curve value for a design speed based on the maximum superelevation and maximum side friction factor. Excessively sharp radii would require superelevation beyond the limits for safe or comfortable operation. Minimum curvature values are sufficient to provide a margin of safety against vehicle rollover and skidding.

The "basic curve equation" (Equation 3-6) governs vehicle operation on a horizontal curve.

$$\frac{0.01e+f}{1-0.01ef}=\frac{v^2}{gR}=\frac{0.067V^2}{R}=\frac{V^2}{15R}$$

where:

e = rate of roadway superelevation, percent

f = side friction (demand) factor

v = vehicle speed, ft/s

g = gravitational constant, 32.2 ft/s^2

V = vehicle speed, mph

R = radius of curve measured to a vehicle's center of gravity, ft

The following equation (Equation 3-8) can be used to calculate the minimum radius of curvature, R_{min} from the "simplified curve formula".

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

Grades

Superelevation adjustments should be considered for grades steeper than 5 percent due to side friction demand. This demand is the result of braking forces on downgrades and tractive forces on steep upgrades.

For rural highways, urban freeways, and high speed urban streets, a balanced design of superelevated, successive horizontal curves is desired to provide a smooth transition.
Maximum side friction factors vary from 0.14 at 50 mph to 0.08 at 80 mph.

On low-speed urban streets, superelevation on horizontal curves may be minimized with lateral forces being sustained by side friction only. Various factors that make superelevation impractical for low-speed urban areas include: *wide pavement areas; adjacent property grades; surface drainage; low-speed operation concerns;* and *intersection frequency.*

Turning Roadways

On turning roadways for right-turning vehicles, the minimum radii should be measured from the inner edge of the traveled way. The radius and superelevation rate is determined from design speed and other values from AASHTO's tables. Sharper curves will have shorter lengths and less opportunity for large superelevation rates. The desirable turning speed is the average running speed of traffic approaching the turn. Maximum superelevation values should be used when possible on ramps to prevent skidding or overturning.

Compound curves can be used exclusively for turning roadways with maximum design speeds of 45 mph. Higher design speeds make this exclusive use impractical due to the large amounts of right-of-way required and should include a mix of tangents and curves.

Transition Design Controls

Horizontal curve safety is determined by a number of factors, such as: curve length; curve radius; spiral transitions; and roadway superelevation. Crashes are more likely on curves with insufficient superelevation or small radii while spiral transitions decrease these chances. Types of transition section designs include:

Superelevation transition

- transitions in the roadway cross slope
- consists of superelevation runoff section for outside-lane cross slope changes (flat to full superelevation); and tangent runout section

Alignment transition

- transitional curves in the horizontal alignment
- spiral or compound curve may be used

Control runoff lengths are typically within the 100 to 650 ft range. This runoff is commonly determined as a function of the slope of the outside edge of the traveled way relative to the roadway centerline profile.

TANGENT-TO-CURVE TRANSITIONS

A "tangent-to-curve" is defined as locations where transition curves are not used and the roadway tangent directly joins the main circular curve. The length of tangent needed for transitioning from adverse crown to full superelevation is called **superelevation runoff**. Superelevation runoff lengths are based on a maximum acceptable difference between the longitudinal grades of the axis of rotation (alignment centerline or pavement reference lines) and the pavement edge. The grade difference (relative gradient) should be limited to a maximum value of 0.50 percent or a longitudinal slope of 1:200 at 50 mph. Greater slopes may be used for design speeds less than 50 mph.

Maximum relative gradients provide shorter runoff lengths at lower speed and longer lengths at higher speeds. Relative gradients of 0.78 and 0.35 percent provide adequate runoff lengths for 15 and 80 mph (Table 3-15).

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

<u>Minimum Length of Runoff</u>

AASHTO recommends using minimum lengths of runoff based on design speed, superelevation, and roadway width. The following equations (Equation 3-23) can be used for rotation about any pavement reference line containing a rotated width (wn1) with a common rate of superelevation and rotated as a plane.

Metric

$$L_r = \frac{(wn_1)\, e_d}{\Delta}(b_w)$$

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

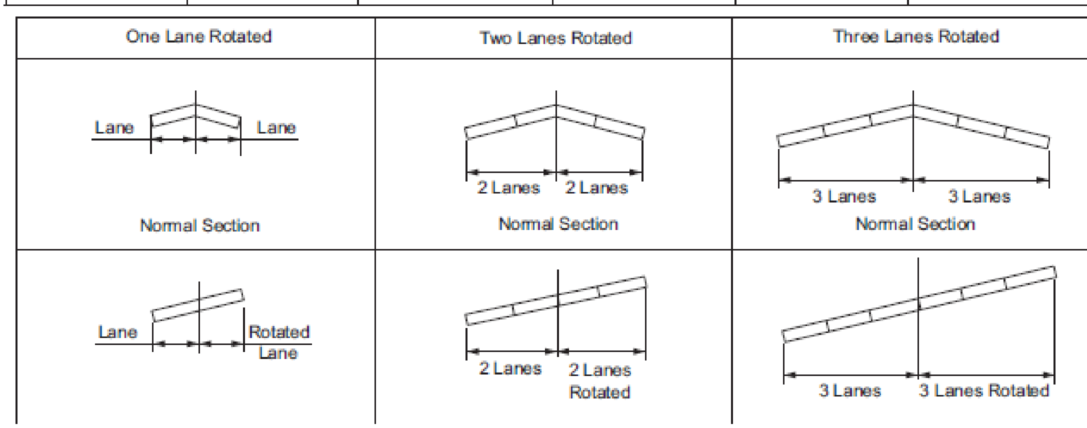
$$L_r = \frac{(wn_1)\, e_d}{\Delta}(b_w) \qquad (3\text{-}23)$$

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

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



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

Minimum Length of Tangent Runoff

The minimum length is determined by the adverse cross slope and its rate of removal. Ideally, this rate should equal the relative gradient for superelevation runoff length.

$$L_t = \frac{e_{NC}}{e_d} L_r$$

where:

L_t = minimum length of tangent runout, ft

e_{NC} = normal cross slope rate, percent

e_d = design superelevation rate, percent

L_r = minimum length of superelevation runoff, ft

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				Design Speed (mph)	Portion of Runoff Located prior to the Curve			
	Number of Lanes Rotated					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

Location With Respect to End of Curve

Typical superelevation runoff length location with respect to the Point of Curvature (PC) is divided between the tangent and curve sections avoiding placement of the entire length on either section. Locating a portion on the tangent is the preferable method since it minimizes peak lateral acceleration and side friction demand, plus it is consistent with the natural spiral path during curve entry. The theoretical proportion of runoff length for tangent sections within the 70 to 90 percent range (with a 10% deviation) produces the best operating conditions.

SPIRAL CURVES

Spiral curves (constantly changing radius) can be used to transition from normal tangent to full superelevation. Common uses involve simple spirals (connecting tangent and circular curves) and combining spirals (connecting curves of different radii). Although using spirals may be more complex, they provide excellent operational capabilities – especially for high speed alignments. Transition curves provide: natural turning paths; gradual changes in lateral forces; minimal lane encroachment; and uniform speeds

Length of Spiral

Many entities use the following equation (Equation 3-25) for calculating the minimum length of a spiral curve.

$$L = \frac{3.15V^3}{RC}$$

where:

L = minimum length of spiral, ft

V = speed, mph

R = curve radius, ft

C = rate of increase of lateral acceleration, ft/s^3

A more realistic value would be that it should equal the length required for superelevation runoff.

Maximum Radius for Use of a Spiral

AASHTO recommends that maximum radii for use of a spiral should be based on a minimum lateral acceleration rate of 4.25 ft/s^2 which produces a range of values such as:

Design Speed	Maximum Radius
15 mph to 80 mph	114 feet to 3238 feet

Minimum Length of Spiral

Spiral curve length is a crucial design control. Optimal operational conditions occur when the spiral length is equivalent to the natural spiral path length used by motorists. The minimum length for a spiral curve is typically based on the following two criteria:

1) Driver comfort **2) Lateral vehicle shifts**

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

$$L_{s,min} = \sqrt{24(p_{min})R}$$

or

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

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)

Maximum Length of Spiral

Research has shown that there is a need to limit spiral lengths in order to prevent driver confusion about the oncoming curve.

$$L_{s,max} = \sqrt{24(p_{max})R}$$

where:

$L_{s,max}$ = maximum length of spiral, ft

p_{max} = maximum lateral offset between the tangent and circular curve (3.3 ft)

R = radius of circular curve, ft

Desirable Length of Spiral

The desirable length of spiral transition curves correspond to 2.0 seconds of travel time (represents natural spiral paths) at roadway design speed. AASHTO provides a table of desirable lengths (Table 3-21).

Design Speed	Spiral Length
15 mph to 80 mph	44 feet to 235 feet

If the table value is less than the minimum spiral curve length (Equations 3-26 and 3-27) – use the minimum length for design.

Length of Superelevation Runoff

Superelevation runoff is recommended to be placed over the spiral length. The change in cross slope starts at a tangent runout section prior to the spiral. The superelevation transitions throughout the spiral to full super for the circular curve.

Length of Tangent Runout

Tangent runout lengths for spiral transitions are similar to the principle for tangent-to-curve transition design. A smooth pavement edge profile with a common edge slope gradient throughout the super runout and runoff sections is preferred.

$$L_t = \frac{e_{NC}}{e_d} L_S$$

where:

L_t = length of tangent runout, ft

L_S = length of spiral, ft

e_d = design superelevation rate, percent

e_{NC} = normal cross slope rate, percent

Location With Respect to End of Curve

The superelevation runoff length should equal the spiral length for the Tangent-to-Spiral (TS) transition (beginning of circular curve) and the Spiral-to-Curve (SC) transition (end). The spiral and superelevation runoffs are integrated between the TS and SC with the roadway rotated to reach full superelevation at the SC.

Methods of Attaining Superelevation

- Revolving traveled way with normal cross slopes about centerline profile (most widely used). Change in elevation of traveled way is done with little distortion (half change in elevation at each edge).
- Revolving traveled way with normal cross slopes about inside-edge profile. Inside-edge profile is shown a line parallel to the profile reference line (preferable for many drainage situations).
- Revolving traveled way with normal cross slopes about outside-edge profile. Similar to inside-edge method except elevation change is attained below the outside-edge profile.
- Revolving traveled way with straight cross slopes about outside-edge profile. Most often used for two-lane one-way roads at locations where the axis of rotation coincides with edge of traveled way adjacent to the median.

Minimum Transition Grades

Potential surface drainage problems in superelevation transitions include:

Lack of adequate longitudinal grade → Poor pavement surface drainage (curb sections)
Negligible cross slope during pavement rotation → Insufficient lateral pavement drainage

Techniques to alleviate these potential drainage problems include:

- Maintaining a minimum vertical grade of 0.5% through the transition
- Maintaining minimum edge-of-pavement grades of 0.2% (0.5% curbed streets) through the transition

Turning Roadways

Drivers naturally follow transitional travel paths. This is best accomplished by using transition curves either between a tangent and circular curve, or between two circular arcs. Spiral lengths for intersection curves are determined in the same method as for typical roadways. The lengths may be shorter than highway curves since motorists accept more rapid changes in these situations. These minimum lengths of spirals are determined by design speed and may range from:

Design Speed	Design Minimum Spiral Length
20 mph to 45 mph	70 feet to 200 feet

Compound Circular Curves

Compound circular curves can produce effective geometries for intersection and interchanges. A smaller radii difference should be used where possible. A desirable maximum value is 1.75:1. For ratios greater than 2:1, a suitable spiral/arc should be placed between the two curves.

OFFTRACKING

Offtracking occurs when a vehicle's rear wheels do not precisely follow the same path as the front wheels when turning or traveling through a horizontal curve. Offtracking is dependent on curve/turn radii, number and location of articulation points, and vehicle wheelbase lengths.

Curve without superelevation (low speed)> rear wheels track inside front wheels
Superelevated curve> rear wheels may track inside front wheels (more or less)
High speeds> rear wheels may track outside front wheels

The amount of widening on horizontal curves for offtracking depends on curve radius design vehicle characteristics (track width for passing, lateral vehicle clearance, width of inner lane vehicle front overhang, rear overhang width, curve difficulty allowance width). This amount increases with design vehicle size and decreases with increasing curve radii.

Traveled-Way Widening on Horizontal Curves

Sometimes, roadways with horizontal curves need to be widened in order to obtain operational conditions similar to tangent sections. The primary reasons for widening on certain curves are:

1. Design vehicle offtracks (rear wheels track inside front wheels) when traversing horizontal curves
2. Drivers difficulty in maintaining vehicles in center of lane

The traveled-way width is dependent on many of the same variables as for offtracking: *track width for passing/meeting vehicles; lateral vehicle clearance; width of inner lane vehicle front overhang;* and *curve difficulty allowance width.*

Normally, the design vehicle should be a truck since offtracking is much greater for them versus passenger cars – **WB-62** is considered appropriate for two-lane open highways.
AASHTO recommends a minimum widening of **2.0 feet** due to economic reasons. Widening on a two-lane, one-way riding surface of a divided highway should be equivalent to a two-lane, two-way highway.

Traveled-way widening on horizontal curves should produce gradual transitions to the curve for smooth alignments of the roadway edges and fit the paths of entering /exiting vehicles. The following guidelines are for both ends of highway curves:

- Widening should be only on the inside edge for simple curves.
- For curves with spirals, widening should be applied to the inside edge or equally divided by the centerline.
- Curve widening transitions should be gradual and sufficient for the traveled way to be fully usable.
- Changes in width are transitioned over a distance of 100 to 200 feet.
- The edge of traveled way should have a smooth and graceful appearance throughout the widening transition.
- For roadways without spirals, one-half to two-thirds of the transition length should be based along the tangent section. For roads with spirals, the width increase is distributed along the spiral length.
- Widening sections may be fully detailed on the roadway construction plans.

<u>Turning Roadway Width Criteria (Intersections)</u>

- Vehicle type: based on size and frequency of vehicle expected
- Curve radii: in addition to vehicle track width determine roadway width
- Expected speed

Turning roadways are classified by: *number of lanes; opportunity for passing;* and *one-way or two-way.*

<u>Design Methods for Turning Roadways</u>

Case I One-lane, one-way operation – no passing provision
Used for minor turning movements, moderate turning volumes, short connecting roadway
Remote chance of vehicle breakdown

Case II One-lane, one-way operation – passing provision
Widths are sufficient for all turning movements of moderate to heavy traffic volumes within capacity of single-lane connection
For breakdowns, low traffic can be maintained

Case III Two-lane, either one or two-way
Two lanes needed for traffic volume

<u>Traffic Conditions for Turning Roadway Widths</u>

Traffic Condition A Predominantly Passenger Car (P) vehicles, some Single-Unit Trucks (SU-30)

Traffic Condition B Majority of Single-Unit Trucks (SU-30), some tractor-semitrailer combination trucks (WB-40) 5 to 10%

Traffic Condition C Predominantly tractor-semitrailer combination (WB-40)

For turning roadways, their width includes shoulders or lateral clearance outside the traveled way. Shoulder widths may vary from none (curbed urban streets) to 2 feet or more (highways). For roadways without curbs or with sloping curbs, adjacent shoulders should match those of the approaches.

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

Turning Roadway Condition	Metric: Shoulder Width or Lateral Clearance Outside of Traveled-Way Edge (m)		U.S. Customary: 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.

If roadside barriers are present, shoulder widths should be measured to the face of barrier (additional graded width of 2.0 feet). For other than low-volume roadways, right shoulders should be stabilized a minimum of 4.0 feet.

Sight Distance – Horizontal Curves

Sight distance across the inside of curves is a crucial design control for any roadway horizontal alignment. For horizontal alignments, the sight line is a chord of the curve. The **stopping sight distance** is along the centerline of the curve's inside lane.

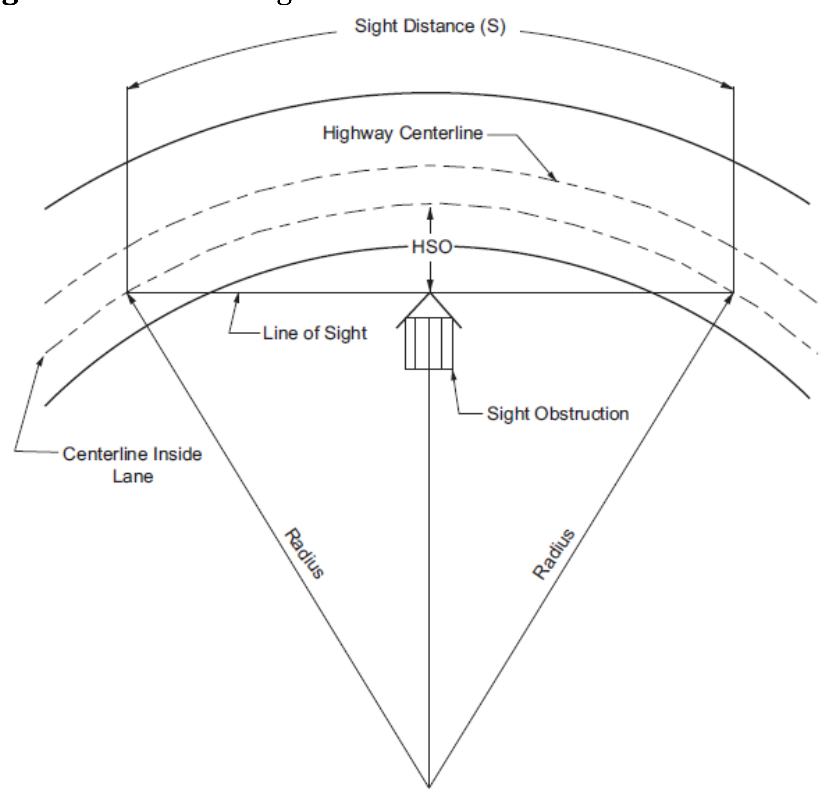


Figure 3-23. Diagram Illustrating Components for Determining Horizontal Sight Distance

Eye Height:	3.50 feet	
Object Height:	2.00 feet	Stopping sight distance
	2.75 feet	Midpoint of sight line where cut slope obstructs sight

Equation 3-36 can be used for circular curve lengths greater than the sight distance for the particular design speed.

$$HSO = R\left[1 - \cos\left(\frac{28.65S}{R}\right)\right]$$

where:

HSO = Horizontal sight line offset, ft

S = Stopping sight distance, ft

R = Radius of curve, ft

At locations where adequate stopping sight distance is not available, the following alternatives may be used: *increase offset to sight obstructions; increase curve radii; or reduce design speed.*

Minimum values for **passing sight distance** (two-lane road) are approximately twice the minimum stopping sight distance.

Eye Height:	3.50 feet	
Object Height:	3.50 feet	Passing sight distance

Due to differences in sight line and stopping sight distance, design for passing sight distance should be limited to tangents and flat curves.

GENERAL CONTROLS

- Any roadway alignment should be directional as possible.
- Minimum radius values should be avoided whenever possible.
- Consistent roadway alignment is desirable.
- Horizontal curves should be long enough for aesthetic purposes.
- Sharp curves should be avoided on lengthy high embankments.
- Exercise caution when using compound circular curves.
- Sudden reversals in alignment should be avoided.
- Avoid “broken-back” or “flat-back” curve arrangements, where possible.
- Horizontal alignment should be carefully coordinated with the roadway profile.
- Avoid changing median widths on tangent alignments.

VERTICAL ALIGNMENT

A roadway's vertical alignment is comprised of crest and sag curves, and the straight grades connecting them. Geometric design of the proposed roadway profile is related to safety, vehicle operations, drainage, and construction issues. The type of terrain to be traversed also plays a major role in the alignment of roadways – particularly the profile. It can be classified into the following categories:

Level: Both horizontal and vertical sight distances are lengthy without difficulty
Rolling: Natural slopes rise or fall below the roadway with occasional steep slopes
Mountainous: Abrupt changes in ground elevation with respect to the roadway

Tangent Grades

Roadway design should encourage uniform operation throughout the proposed facility. Nearly all passenger cars can readily negotiate grades as steep as 4 to 5 percent without a significant loss in speed. Speeds decrease progressively with grade increases.

Vertical grades have a greater effect on truck speeds as opposed to passenger cars. Tangent grades balance construction costs with desired operations. Lengthy grades greater than 3% start to influence passenger car speeds while shorter, steeper grades affect truck speeds. Although the average speeds of trucks and passenger cars are similar for level roadway sections, trucks usually increase downgrade speeds up to 5% and decrease upgrade speeds by 7%. Their maximum speed is dependent on the length/steepness of the grade and the truck's weight/power ratio (gross vehicle weight divided by the net engine power).

Travel times and speeds for trucks are byproducts of the weight/power ratio. Presently, acceptable values for highway users are approximately **200 lb/hp**. These values have been steadily decreasing over the years resulting in greater power and better climbing ability on upgrades.

Typically, vertical grades should be less than the maximum design grade. Design guidelines for maximum grades have been established from grade controls presently in use – however, these maximum rates should be rarely used, if possible.

Design Speed	Maximum Grade
70 mph	5%
30 mph	7 to 12% (depending on terrain)

On major routes, maximum grade values of 7 to 8 percent are commonly used for 30 mph design speed. In most cases, grades should be less than the maximum design grade.

However, for one-way downgrades less than 500 ft long, the maximum grade should be approximately 1% steeper than other locations. The maximum may be 2% steeper for low-volume rural highways.

Maintaining adequate minimum grades is a primary concern in many locations. For roadways with adequate cross slopes for surface drainage, a typical value for minimum grade is 0.5 percent. The vertical profile may affect road drainage by creating very flat roads/sag curves that may have poor drainage, or steep roads with high velocity flows.

0.3 percent	*Minimum control in some states*
0.15 percent	*Practical minimum for flat terrain*
0 percent (flat)	*Should be avoided – relies totally on roadway cross-slope for drainage*

Critical Length of Grade is the maximum length of upgrade on which a loaded truck may operate without an unacceptable speed reduction. Research has shown that the more a vehicle deviates from the average roadway speed, the greater its chances of crashing. The following data may be used to determine values for critical lengths of grade:

– Size , power, and gradeability data for design vehicle
  A typical loaded truck used as a design control has a weight/power ratio of 200 lb/hp
– Entrance speeds to critical length
  The average running speed can be used for vehicle speeds at the beginning of an uphill approach
– Minimum tolerable speeds of trucks on upgrades
  Roadways should be designed to prevent intolerable truck speed reductions for following drivers

A **Climbing Lane** is an added lane for slow-moving vehicles (uphill) so other vehicles may use the normal roadway lanes to pass. These lanes may be used for locations where a level of service or truck speed is much less on an upgrade versus the approach. Climbing lanes are an inexpensive way to overcome capacity reductions, improve operation in truck congestion areas, and reduce crashes.

Justification Criteria for Climbing Lanes

- Upgrade traffic flow rate exceeds 200 vehicles per hour
- Upgrade truck flow rate exceeds 20 vehicles per hour
- One of the following exists:

    10 mph or greater speed reduction expected by heavy trucks
    Level of service of E or F
    Reduction of 2 or more levels of service from approach segment to grade

Successful methods for increasing passing opportunities on 2-lane roads include: passing lanes; turnouts; shoulder driving; and shoulder driving.

A P**assing Lane** is an additional lane to improve traffic operations in low capacity sections. A minimum sight distance of 1000 feet is recommended for taper approaches. The optimal lane length is typically 0.5 to 2 miles with longer lane lengths for higher traffic volumes. Transition tapers at the ends of added-lane sections can be designed from the following equations (Equations 3-37 and 3-38):

$$L = WS$$ 45 mph or greater

$$L = \frac{WS^2}{60}$$ Less than 45 mph

where:

L = Length of taper, ft

W = Width, ft

S = Speed, mph

Table 3-31. Optimal Passing Lane Lengths for Traffic Operational Efficiency (*28, 29*)

Metric		U.S. Customary	
One-Way Flow Rate (veh/h)	Passing Lane Length (km)	One-Way Flow Rate (veh/h)	Passing Lane Length (mi)
100–200	0.8	100–200	0.50
201–400	0.8–1.2	201–400	0.50–0.75
401–700	1.2–1.6	401–700	0.75–1.00
701–1200	1.6–3.2	701–1200	1.00–2.00

A **Turnout** is a widened, unobstructed shoulder area for slow-moving vehicles to pull out of traffic (low volume roads, difficult terrain, etc.).

Typical entry & exit taper lengths:	*50 to 100 feet*	
Minimum turnout width:	*12 feet*	*16 feet (desirable)*
Minimum sight distance:	*1000 feet on approach*	

Shoulder driving is a practice where slow-moving vehicles move to the shoulder when approached from the rear by another vehicle, and returns to the roadway after being passed. This custom occurs where there are adequate paved shoulders that function as continuous turnouts. Shoulder widths should be a minimum of 10 feet, with 12 feet as a desirable value.

Shoulder use sections are segments where slow vehicles are permitted to use paved shoulders at specific signed sites (similar to shoulder driving). This application is a more limited use of paved shoulders. Lengths typically range from 0.2 to 3 miles with minimum widths of 10 feet (preferably 12 feet).

VERTICAL CURVES

A road's vertical alignment consists of road slopes (grades) connected by vertical curves. Vertical curves (parabolic) provide a gradual change from one grade to another for vehicles to smoothly navigate any grade changes. Normally, a parabolic curve with an equivalent vertical axis is centered on the Vertical Point of Intersection (VPI). These curves are either classified as **sag** or **crest**.

Design guidelines for vertical curve lengths are generally based on providing for sufficient sight distance and driver comfort. Longer stopping sight distances should be used where possible.

The **K value** is the horizontal distance needed to create a one-percent change in gradient. It is a measure of curvature and is expressed as the ratio of the vertical curve length to the algebraic difference in the grades (L/A). This is useful in determining the horizontal distance from the Vertical Point of Curvature (VPC) to the sag or crest points. The K value can also be useful for determining minimum vertical curve lengths.

Terms

A = algebraic difference in grades (percent)
L = vertical curve length
VPC = begin of vertical curve
VPT = end of vertical curve
G1 = initial roadway (tangent) grade
G2 = final roadway (tangent) grade
h₁= Height of eye above roadway
h₂= Height of object above roadway
L = vertical curve length
VPI = point of vertical interception (intersection of initial and final grades)

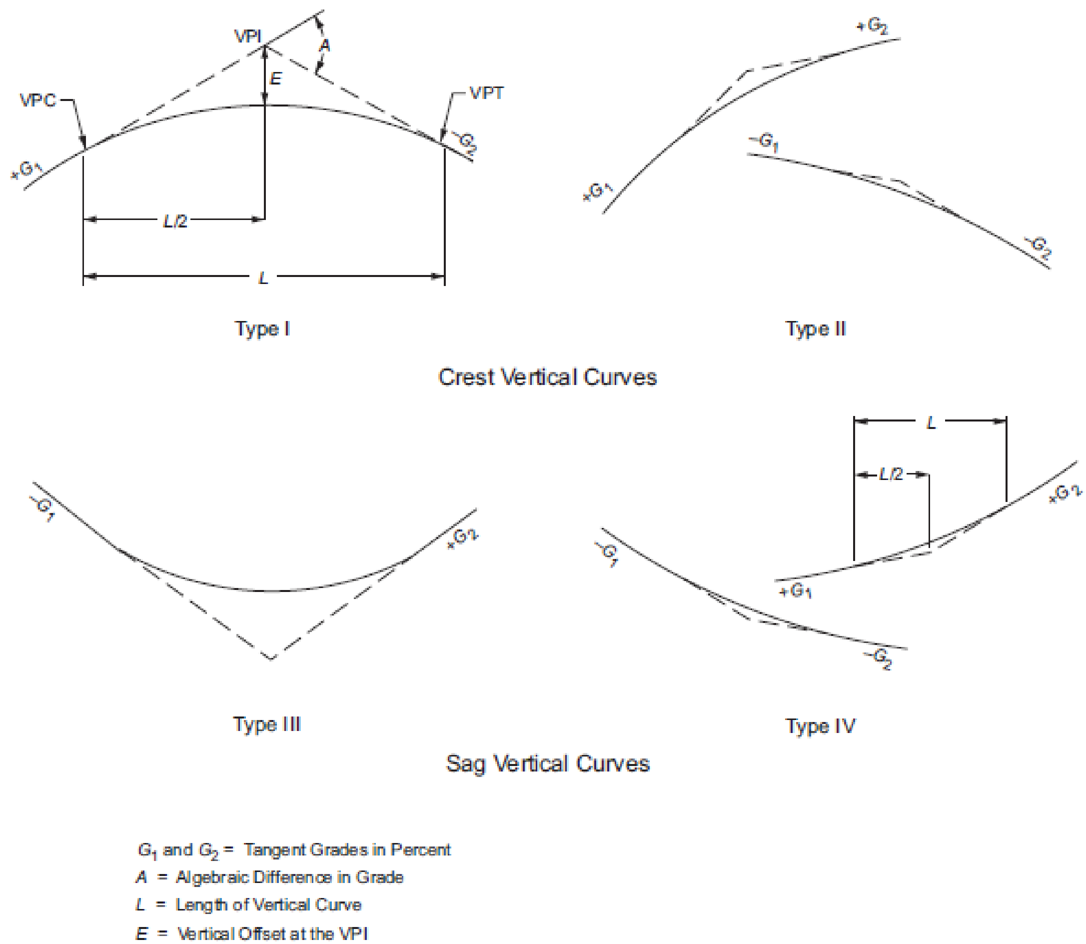


Figure 3-41. Types of Vertical Curves

SAG VERTICAL CURVES

Sag vertical curves have a tangent slope at the end of the curve which is higher than at the beginning. Sag curves appear as valleys by first going downhill, reaching the bottom of the curve, and continuing uphill (resembling an upward concave curve).

Sag Vertical Curve Criteria

Headlight sight distance   Passenger comfort   Drainage control   General appearance

The most important determinant of sag curve length is **headlight sight distance**. When traveling on a sag curve at night, this sight distance is limited by the headlight position and direction. This distance must be adequate for the driver to see a roadway obstruction and stop within the headlight sight distance. The sag curve length should be of sufficient length for the light beam distance to be approximately equal to the stopping sight distance in order for drivers to see the roadway ahead.

Headlight Sight Distance Assumptions

Headlight height: 2 feet

Upward divergence of light beam from longitudinal axis of vehicle: 1 degree

$$L = \frac{AV^2}{46.5}$$

where:

L = length of sag vertical curve, ft

A = algebraic difference in grades, percent

V = design speed, mph

The vertical curve length needed to satisfy passenger comfort is typically 50% of the needed headlight sight distance for normal conditions. A good rule-of-thumb approximation for minimum sag vertical curve length is 100A or K = 100 feet per percent change in grade.

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*.

CREST VERTICAL CURVES

Crest vertical curves have a lower tangent slope at the end of the curve than at its beginning. Crest vertical curves are convex upwards and appears as a hill, with vehicles first going uphill, reaching the top of the curve and then continuing downhill. A crucial design criterion for these curves is **stopping sight distance** (the distance a driver can see over the curve crest). This distance is determined by the speed of roadway traffic. The appropriate equation depends on the length of the vertical curve versus the available sight distance. Equations 3-41 and 3-42 are the basic formulas for determining crest vertical curve lengths in terms of algebraic grade difference (A) and sight distance (S).

When S is less than L,

$$L = \frac{AS^2}{100\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2}$$

When S is greater than L,

$$L = 2S - \frac{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}{A}$$

where:

L = length of vertical curve, ft

A = algebraic difference in grades, percent

S = sight distance, ft

h_1 = height of eye above roadway surface, ft

h_2 = height of object above roadway surface, ft

Stopping Sight Distance Criteria

Height of Eye:	3.50 feet
Height of Object:	2.00 feet

Using the typical values for eye height and object height, the calculations for crest vertical curve lengths become the following equations (Equations 3-43 and 3-44).

When S is less than L,

$$L = \frac{AS^2}{2158}$$

When S is greater than L,

$$L = 2S - \frac{2158}{A}$$

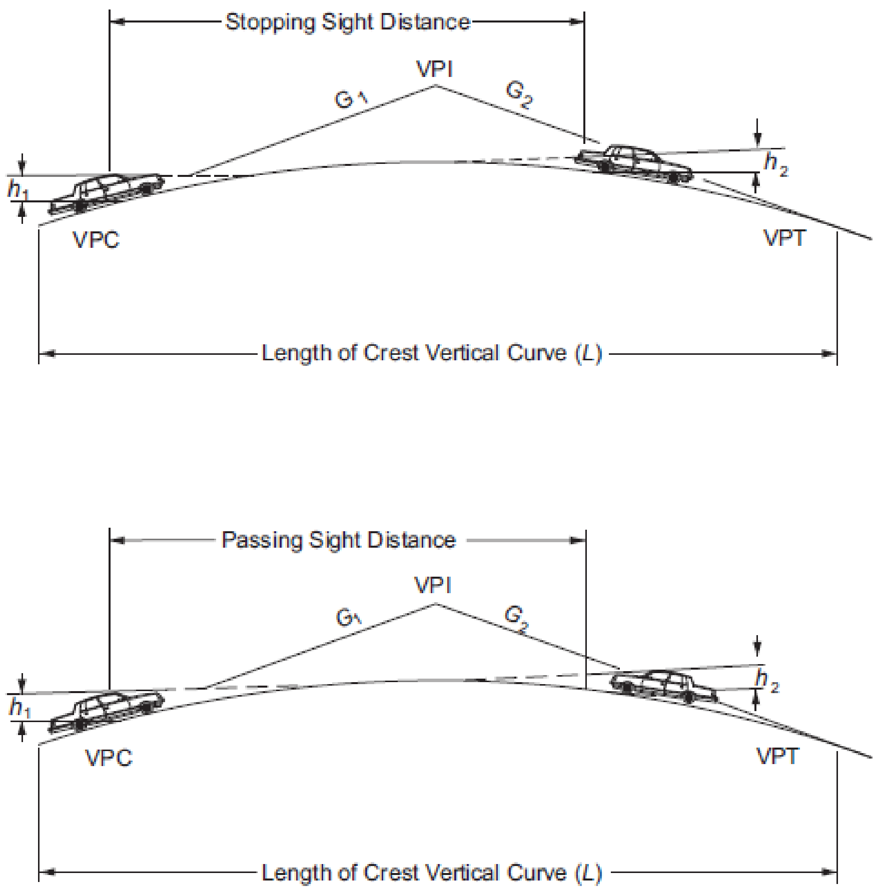


Figure 3-42. Parameters Considered in Determining the Length of a Crest Vertical Curve to Provide Sight Distance

Crest vertical curve design values for **passing sight distance** differ from stopping sight distance due to sight distance and object heights (3.50 feet) and may be determined from the following formulas (Equations 3-45 and 3-46).

When S is less than L,

$$L = \frac{AS^2}{2800}$$

When S is greater than L,

$$L = 2S - \frac{2800}{A}$$

Using passing sight distance criteria, minimum crest curve lengths are much longer than those for stopping sight distance. Typically, it is not realistic to use passing sight distance controls due to high costs of crest cuts and difficulty of integrating long vertical curves to the topography.

Passing sight criteria for crest curves may be appropriate for: *low speed roadways with gentle grades; high speeds with small grade differences;* and *locations not needing significant grading.*

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), $K = L/A$.

General Controls for Vertical Alignments

- Use a smooth, gradual grade consistent with roadway type and terrain
- Avoid hidden dips/changes in the roadway profile
- Evaluate any proposed profile containing substantial momentum grades with traffic operations
- Avoid "broken back" (consecutive vertical curves in the same direction) gradelines
- Place steep grades at the bottom and flatter grades near the top of ascents
- Reduce grades through at-grade intersections with moderate to steep grades
- Avoid sag curves in cuts, where possible

COORDINATION OF HORIZONTAL AND VERTICAL ALIGNMENTS

Roadway geometry influences its safety performance. Research has shown that roadway factors are the **second** most contributing factor to road accidents. Crashes tend to occur more frequently at locations with sudden changes in road character (example: sharp curves at the end of long tangent roadway sections). The concept of **design consistency** compares adjacent road segments and identifying sites with changes that might appear sudden or unexpected. Design consistency analysis can be used to show the decrease in operating speed at a curve.

The horizontal and vertical geometries are the most critical design elements of any roadway. These alignments should be integrated to enhance vehicle operation, uniform speed, and facility appearance without additional costs (examples: checking for additional sight distance prior to major changes in the horizontal alignment; or revising design elements to eliminate potential drainage problems). Computer-aided design and drafting (CADD) is commonly used to facilitate the iterative three-dimensional design and produce an optimal coordination of horizontal and vertical alignments.

Design speed helps to determine roadway location and keeps all design elements (traffic, topography, geotechnical concerns, culture, future development, project limits, etc.) in balance. It limits many design values (curves, sight distance) and influences others (width, clearance, maximum gradient).

AASHTO provides the following general design guidelines regarding horizontal and vertical alignment combinations:

- Vertical and horizontal elements should be balanced to produce a design which optimizes safety, capacity, operation, and aesthetics within the location's topography.
- Avoid sharp horizontal curves near the top of a crest vertical curve or near the low point of a sag vertical curve. Using higher design values (well above the minimum) for design speed can produce suitable designs and meet driver's expectations.
- Horizontal and vertical curves should be flat as possible for intersections with sight distance concerns.
- For divided roadways, it may be suitable to vary the median width or use independent horizontal/vertical alignments for individual one-way roads.
- Roadway alignments should be designed to minimize nuisances in residential areas. Typical measures may include: depressed facilities (decreases facility visibility and noise); or horizontal adjustments (increases buffer zones between traffic and neighborhoods).
- Horizontal and vertical elements should be used to enhance environmental features (parks, rivers, terrain, etc.). Roadways should lead into outstanding views or features instead of avoiding them where possible.

Exception

Long tangent sections for sufficient passing sight distance may be appropriate for two-lane roads needing passing sections at frequent intervals.

REFERENCES

A Policy on Geometric Design of Highways and Streets, 6th Edition.
AASHTO. Washington, D.C. 2011.

Note: All figures, illustrations, tables, etc. contained within this course are from this text unless noted otherwise.

Civil Engineering Reference Manual, 8th Edition.
Michael R. Lindeburg. Professional Publications, Inc.
Belmont, CA 2001.

Flexibility in Highway Design.
Federal Highway Administration. Washington, D.C. 1997.

Handbook of Simplified Practice for Traffic Studies.
Center for Transportation Research & Education – Iowa State University.
Ames, Iowa. 2002.

Manual on Uniform Traffic Control Devices, 2009 Edition.
Federal Highway Administration. Washington, D.C. 2009.

Mini-Roundabouts Technical Summary.
FHWA. Washington, D.C. 2010.

NCHRP Report 672 Roundabouts: An Informational Guide, 2nd Edition.
FHWA. Washington, D.C. 2010.

Roadside Design Guide, 4th Edition.
AASHTO. Washington, D.C. 2011.

Roundabouts: An Informational Guide.
FHWA. Washington, D.C. 2000.

Standard Roadway Drawings.
Tennessee Department of Transportation.

Traffic Engineering Handbook, 5th Edition.
Institute of Transportation Engineers. Washington, D.C. 1999.

Traffic Engineering Handbook, 6th Edition.
Institute of Transportation Engineers. Washington, D.C. 2009.