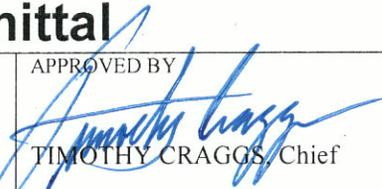


manual change transmittal

TITLE DIVISION OF DESIGN HIGHWAY DESIGN MANUAL SIXTH EDITION – CHANGE 07/01/15		APPROVED BY  TIMOTHY CRAGGS, Chief	NO.
			Date Issued: 06/29/15 Page 1 of 4
SUBJECT AREA Table of Contents; List of Figures; List of Tables; Chapters: 60, 80, 100, 300, 1000; and, Index		ISSUING UNIT DIVISION OF DESIGN	
SUPERCEDES SEE BELOW FOR SPECIFIC PAGE NUMBERS		DISTRIBUTION ALL HOLDERS OF THE 6TH EDITION, HIGHWAY DESIGN MANUAL	

The Foreword; Table of Contents; List of Figures; List of Tables; Chapters: 60, 100, 200, 300, 400, 500, 810, 820, 830, 900, 1000; and the Index of the Sixth Edition, Highway Design Manual (HDM) have been revised. The changes to the HDM are summarized below with change sheets available on the Department Design website at: <http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm>. Changes include revised pedestrian refuge island guidance, highway drainage design, and planting guidance for freeways, expressways and conventional highways. Also included are clarifications of right-turn channelization guidance, loop on-ramp guidance, and Class I bikeway guidance as well as revisions that reflect current nomenclature and other errata. These changes are effective July 1, 2015, and shall be applied to on-going projects in accordance with HDM Index 82.5 – Effective Date for Implementing Revisions to Design Standards.

HDM Holders are encouraged to use the most recent version of the HDM available on-line at the above website. Should a HDM Holder choose to maintain a paper copy, the Holder is responsible for keeping their paper copy up to date and current. Using the latest version available on-line will ensure proper reference to the latest design standards and guidance. If you would like to be notified automatically of any significant changes or updates to the HDM, go to <http://www.dot.ca.gov/hq/oppd/hdm/hdmlist.htm>.

A summary of the most significant revisions are as follows:

- Foreword** **Common Conversion Factors to Metric**
 Corrected both Metric and U.S. Customary (English) mass conversion factors.
- Index 62.5** **Landscape Architecture, Page 60-6**
 Revised highway planting definition. Corrected typographical error in roadside rehabilitation reference to the National Pollution Discharge Elimination System.
- Index 62.7** **Pavement, Page 60-8**
 Deleted safety edge definition. Minor grammatical corrections made elsewhere.
- Table 82.1A-C** **Standards, Page 80-13**
 The tables summarizing mandatory and advisory standards as well as decisions requiring other approvals have been revised, consistent with this publishing.
- Index 110.10** **Proprietary Items, Page 100-30**
 Corrected reference identifying the approval authority over the use of proprietary items.
- Figure 202.5B** **Superelevation Transition Terms & Definitions, Page 200-14**
 Corrections to superelevation terms as well as elements shown on a right curve.

<u>Index 203.4</u>	Curve Length and Central Angle, Page 200-16 Clarification of curve requirements for central angles larger than 30 minutes.
<u>Index 203.8</u>	Spiral Transitions, Page 200-17 Replaced the word “recommended” with “may” when describing the typical characteristics of the roadway when considering the use of spiral transitions.
<u>Table 204.8</u>	Falsework Span and Depth Requirements, Page 200-24 Minor typographical corrections.
<u>Index 205.4</u>	Driveways, Page 200-27 Corrected the minimum length of paving on driveways from State highways.
<u>Figure 208.10B</u>	Combination Vehicular Barrier and Pedestrian Railings for Bridge Structures, Page 200-42 Corrected the dimension lines indicating sidewalk width on bridge structures.
<u>Index 301.2</u>	Class II Bikeway (Bike Lane) Lane Width, Page 300-1 Minor errata to clarify additional marked lane width as bike lane width.
<u>Index 305.1</u>	Median Standards, Page 300-17 Median pedestrian refuge guidance was moved to Index 405.4(3).
<u>Index 306.1</u>	Right of Way, General Standards, Page 300-19 Corrected spelling of the word “installation”.
<u>Index 402.4</u>	Consider All Users, Page 400-4 Corrected transit stop reference location to Topic 108 for additional information.
<u>Index 403.3</u>	Angle of Intersection, Page 400-5 Clarified the importance of right angle intersections in the opening paragraph.
<u>Figure 403.6B</u>	Bicycle Left-Turn-Only Lane, Page 400-8 Typographical correction of center island cross-hatching pattern.
<u>Index 403.6</u>	Turning Traffic, Page 400-9 Revised who to consult for signing and delineation of bicycle lanes at intersections. Clarified design of intersections at interchanges guidance to reflect intent of reducing speeds at conflict points for motorist, bicyclist, and pedestrian accommodation.
<u>Index 403.9</u>	Effective Signal Control, Page 400-9 Updated text to reflect revised Intersection Control Policy objectives.
<u>Index 404.2</u>	Design Considerations, Page 400-11 Typographical corrections to Sub indexes regarding curb and gutters and sidewalks.
<u>Index 404.4(4)(a)</u>	California Legal, Page 400-13 Minor typographical correction of the word “Department’s”.

<u>Table 405.1B</u>	Application of Sight Distance Requirements, Page 400-23 Clarified note 1 of the table.
<u>Index 405.3</u>	Right-turn Channelization, Page 400-25 Corrected typographical errors and nomenclature. Clarified curve radius guidance to reflect intent of reducing speed at right-turn lanes for motorist, bicyclist, and pedestrian accommodation.
<u>Figure 405.2B</u>	Minimum Median Left-turn Channelization, Page 400-27 Typographical correction of center island cross-hatching pattern.
<u>Index 405.4(3)</u>	Pedestrian Refuge, Page 400-30 The pedestrian refuge advisory standard from former Index 305.1 is included and rewritten to specify use for complete street application. The existing advisory standard addressing the minimum width of traffic islands was upgraded to an absolute requirement per the Americans with Disabilities Act.
<u>Index 501.3</u>	Spacing, Page 500-1 Corrected nomenclature and typographical errors. Updates consistent with revised PDPM Chapter 27 guidance.
<u>Figure 504.2A</u>	Single Lane Freeway Entrance, Page 500-12 Corrected typographical error, Detail “A”, inlet nose dimension should be shown as 3’.
<u>Index 504.3(3)</u>	Location and Design of Ramp Intersections on the Crossroads, Page 500-21 Revised reference to Index 403.6(2) for further guidance related to the design of uncontrolled entries and exits from freeway ramps to local roads.
<u>Index 504.3(8)</u>	Loop Ramps, Page 500-22 Clarified guidance related to pavement structure consideration at freeway ramps with radii less than 300 feet and other design vehicle accommodation design elements.
<u>Figure 504.3J</u>	Location of Ramp Intersections on the Crossroads, Page 500-32 Corrected corner sight distance lengths consistent with Table 405.1A.
<u>Figure 504.3K</u>	Transition to Two-lane Exit Ramp, Page 33 Corrected misspelling of the word “Index” in Notes.
<u>Index 504.5</u>	Auxiliary Lanes, Page 500-36 Auxiliary lane discussion was revised as a result of the pending retirement of Design Information Bulletin 77 including the conversion of an advisory standard to permissive.
<u>Index 504.7</u>	Weaving Sections, Page 500-39 Revised nomenclature.
<u>Index 504.8</u>	Access Control, Page 500-39 Revised the last mandatory standard with the deletion of the phrase “the construction of future”.

- Chapter 810** **Hydrology, Page 810-1**
Multiple revisions throughout the Chapter to update and include additional references, define the concept of stationarity as it relates to climate variability, update the USGS Regional Regression Equations based on more recent studies and expand the guidance on hydrograph development.
- Index 821.5** **Effects of Tide, Storm Surge and Wind, Page 820-3**
Updated guidance for analyzing the combined effects of tides and rainfall runoff in coastal locations in order to determine the 100-year flood event.
- Index 826.3(6)** **Entrance Risers, Page 820-8**
Typographical correction.
- Index 829.7(3)** **Types of Conduit, Page 820-13**
Unit of measure conversion from metric to U.S. Customary units.
- Index 831.4(5)** **Other Considerations, Page 830-5**
Minor correction to reference number that had changed.
- Index 833.2** **Grade, Cross Slope and Superelevation, Page 830-5**
Minor corrections to references and nomenclature that had changed.
- Index 838.5(3)** **Appurtenant Structures, Page 830-20**
Nomenclature change from flap gate to flag drainage gate.
- Index 901.1** **Landscape Architecture Program, Page 900-1**
Updated Landscape Architecture Program responsibilities and approvals.
- Index 901.2** **Cross References, Page 900-1**
Updated and revised landscape architecture references.
- Index 902.1** **General Guidance for Freeways and Expressways, Page 900-1**
General update and partial rewrite of planting guidance for freeways and expressways. Correction of typographical errors and revised nomenclature.
- Index 902.2** **Sight Distance and Clear Recovery Zone Standards for Freeways and Expressways, Page 900-3**
Expanded sight distance and plant setback guidance. Clarified and expanded clear recovery zone guidance related to the planting of large trees including the revised definition of large trees. Clarified the advisory standard related to the planting of large trees on freeways and expressways, including interchange areas. Provided species examples of large and small trees.
- Index 902.3** **Planting Guidance for Large Trees on Conventional Highways, Page 900-4**
Rewritten guidance related to the planting of large trees on conventional highway roadsides and medians. Large tree setback requirements on conventional highway roadside and median advisory and mandatory standards have been relocated onto new Table 902.3.
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<u>Table 902.3</u>	Large Tree Setback Requirements on Conventional Highways, Page 900-5 Large tree setback requirements on conventional highway roadside and median advisory and mandatory standards have been relocated onto new Table 902.3.
<u>Index 902.4</u>	Planting Procedures, Selection and Location, Page 900-6 Relocated portions of former Index 902.3 Planting Guidelines in new Index title including design procedures, plant selection, plant location, planting on or near walls, planting of vines on bridge structures and planting in vicinity of airports and heliports.
<u>Index 902.5</u>	Irrigation Guidelines, Page 900-7 Changed Index number from 902.4 to 902.5, including minor updates.
<u>Index 903.3</u>	Site Selection, Page 900-9 Converted the safety roadside rest area metric unit typographical error to U.S. Customary units.
<u>Index 903.4</u>	Facility Size and Capacity Analysis, Page 900-10 Minor typographical corrections.
<u>Index 1001.1</u>	Bicycle Transportation, Page 1000-1 Corrected spelling of the term non-motorized.
<u>Index 1001.2</u>	Streets and Highways Code References, Page 1000-1 Corrected spelling of the term non-motorized.
<u>Index 1001.3</u>	Vehicle Code References, Page 1000-1 Typographical corrections related to Sub index lettering.
<u>Index 1003.1</u>	Class I Bikeways (Bike Paths), Page 1000-3 Reorganized and clarified existing guidance within the Sub-indexes of Topic 1003. Defined the use of the term “shoulder” as used in the context of a bike path. Added new mandatory standard related to bike path shoulder cross slope to facilitate drainage. Various typographical corrections.
<u>Figure 1003.1A</u>	Two-Way Class I Bikeway (Bike Path), Page 1000-6 Minor figure clarifications to paved width dimension call out and area beyond shoulder.
<u>Figure 1003.1B</u>	Typical Cross Section of Class I Bikeway Parallel to Highway, Page 1000-7 Included conventional abbreviations to various dimension call-outs.
<u>Figure 1003.1C</u>	Minimum Length of Bicycle Path Crest Vertical Curve (L), Page 1000-11 Corrected reference number where bike path stopping sight distance can be found.
<u>Figure 1003.1D</u>	Minimum Lateral Clearance (m) on Bicycle Path Horizontal Curves, Page 1000-12 Corrected reference to Index 1003.1(10).

Metric Basics

Measurable Attribute - Basic Units	Unit	Expression	
Length	meter	m	
Mass	kilogram	kg	
Luminous intensity	candela	cd	
Time	second	s	
Time	hour	h	
Electric current	ampere	A	
Thermodynamic temperature	Kelvin	K	
Amount of substance	mole	mol	
Volume of liquid	liter	L	
Measurable Attribute - Special Names	Unit	Expression	
Frequency of a periodic phenomenon	hertz	Hz (1/s)	
Force	newton	N (kg·m/s ²)	
Energy/work/quantity of heat	joule	J(N·m)	
Power	watt	W (J/s)	
Pressure/stress	pascal	Pa (N/m ²)	
Celsius temperature	Celsius	°C	
Quantity of electricity/electrical charge	coulomb	C	
Electric potential	volt	V	
Electric resistance	ohm	Ω	
Luminous flux	lumen	lm	
Luminance	lux	lx (lm/m ²) or (cd/m ²)	
Measurable Attribute - Derived Units	Unit	Expression	
Acceleration	meter per second squared	m/s ²	
Area	square meter	m ²	
Area	hectare	ha (10 000 m ²)	
Density/mass	kilogram per cubic meter	kg/m ³	
Volume	cubic meters	m ³	
Velocity	meter per second	m/s	
Mass	tonne	tonne (1000 kg)	
Multiplication Factors	Prefix	Symbol	Pronunciations
1 000 000 000 = 10 ⁹	giga	G	jig' a (<i>ias</i> in <i>jig</i> , <i>a</i> as in <i>a</i> -bout)
1 000 000 = 10 ⁶	mega	M	as in <i>mega</i> -phone
1000 = 10 ³	kilo	k	kill' oh
100 = 10 ²	*hecto	h	heck' toe
10 = 10 ¹	*deko	da	deck' a (<i>a</i> as in <i>a</i> -bout)
0.1 = 10 ⁻¹	*deci	d	as in <i>deci</i> -mal
0.01 = 10 ⁻²	*centi	c	as in <i>centi</i> -pede
0.001 = 10 ⁻³	milli	m	as in <i>mili</i> -tary
0.000 001 = 10 ⁻⁶	micro	μ	as in <i>micro</i> -phone
0.000 000 001 = 10 ⁻⁹	nano	n	nan' oh (<i>an</i> as in <i>ant</i>)

* to be avoided where possible

Common Conversion Factors to Metric

Class	Multiply:	By:	To Get:
Area	ft ²	0.0929	m ²
	yd ²	0.8361	m ²
	mi ²	2.590	km ²
	acre	0.404 69	ha
Length	ft	0.3048	m
	in	25.4	mm
	mi	1.6093	km
	yd	0.9144	m
Volume	ft ³	0.0283	m ³
	gal	3.785	L *
	fl oz	29.574	mL *
	yd ³	0.7646	m ³
Mass	acre ft	1233.49	m ³
	oz	28.35	g
	lb	0.4536	kg
	kip (1,000 lbs)	0.4536	453.6 kg
	short ton (2,000 lbs)	907.2	kg
Density	short ton	0.9072	tonne (1000 kg)
	lb/yd ³	0.5933	kg/m ³
	lb/ft ³	16.0185	kg/m ³
Pressure	psi	6894.8	Pa
	ksi	6.8948	MPa (N/mm ²)
	lb _f /ft ²	47.88	Pa
Velocity	ft/s	0.3048	m/s
	mph	0.4470	m/s
	mph	1.6093	km/h
Temp	°F	$t_{°C} = (t_{°F} - 32) / 1.8$	°C
Light	footcandle (or) lumen/ft ²	10.7639	lux (lx) (or) lumen/m ²

* Use Capital "L" for liter to eliminate confusion with the numeral "1"

Land Surveying Conversion Factors

Class	Multiply :	By:	To Get
Area	acre	4046.87261	m ²
	acre	0.404 69	ha (10 000 m ²)
Length	ft	1200/3937**	m

** Exact, by definition of the US Survey foot, Section 8810, State of California Public Resources Code

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- circulates. The central island does not necessarily need to be circular in shape.
- (2) *Circulatory Roadway.* The curved roadbed that users of a roundabout travel on in a counterclockwise direction around the central island.
 - (3) *Channelization.* The separation or regulation of conflicting movements into definite paths of travel by the use of pavement markings, raised islands, or other suitable means to facilitate the safe and orderly movement of vehicles, bicycles and pedestrians.
 - (4) *Crosswalk.* Crosswalk is either:
 - (a) That portion of a roadway included within the prolongation or connection of the boundary lines of sidewalks at intersections where the intersecting roadways meet at approximately right angles, except the prolongation of such lines from an alley across a street.
 - (b) Any portion of a roadway distinctly indicated for pedestrian crossing by lines or other markings on the surface.
 - (5) *Geometric Design.* The arrangement of the visible elements of a road, such as alignment, grades, sight distances, widths, slopes, and other similar elements.
 - (6) *Gore.* The area immediately beyond the divergence of two roadbeds bounded by the edges of those roadbeds.
 - (7) *Grade Separation.* A crossing of two highways, highway and local road, or a highway and a railroad at different levels.
 - (8) *Inscribed Circle Diameter.* The distance across the circle of a roundabout, inscribed by the outer curb (or edge) of the circulatory roadway. It is the sum of the central island diameter and twice the circulatory roadway width.
 - (9) *Interchange.* A system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of vehicles between two or more roadways on different levels.
 - (10) *Interchange Elements.*
 - (a) Branch Connection--A multilane connection between two freeways.
 - (b) Freeway-to-freeway Connection--A single or multilane connection between freeways or any two high speed facilities.
 - (c) Ramp--A connecting roadway between a freeway or expressway and another highway, road, or roadside area.
 - (11) *Intersection.* The general area where two or more roadways join or cross, including the roadway and roadside facilities for movements in that area.
 - (12) *Island.* A defined area between roadway lanes for control of vehicle movements or for pedestrian refuge. Within an intersection a median or an outer separation is considered an island.
 - (13) *Landscape Buffer/Strip.* A planted section adjacent to the legs of a roundabout that separates users of the roadway from users of the shared use/Class I Bikeway and assists with guiding pedestrians to the designated crossing locations. Also known as "way finding."
 - (14) *Minimum Turning Radius.* The radius of the path of the outer front wheel of a vehicle making its sharpest turn.
 - (15) *Offset Left-Turn Lanes.* Left-turn lanes are shifted as far to the left as practical rather than aligning the left-turn lane exactly parallel with and adjacent to the through lane.
 - (16) *Offtracking.* The difference between the paths of the front and rear wheels of a vehicle as it negotiates a turn.
 - (17) *Pedestrian Refuge.* A section of pavement or sidewalk, completely surrounded by asphalt or other road materials, where users can stop before completing the crossing of a road.
 - (18) *Roundabout.* A type of circular intersection with specific geometric and traffic control features that in combination lower speed operations and lower speed differentials among all users immediately prior to, through, and beyond the intersection. Vehicle speed is

controlled by deflection in the path of travel, and the “yield upon entry” rule for traffic approaching the roundabout’s circulatory roadway. Curves and deflections are introduced that limit operating speeds.

- (19) *Splitter Island.* A raised or painted traffic island that separates traffic in opposing directions of travel. They are typically used at roundabouts and on the minor road approaches to an intersection.
- (20) *Skew Angle.* The complement of the acute angle between two centerlines which cross.
- (21) *Swept width.* The total width needed by the vehicle body to traverse a curve. It is the distance measured along the curve radius from the outer front corner of the body to the inner rear corner of the body as the vehicle traverses around a curve. This width is used to determine lane width and clearance to objects, such as signs, poles, etc., as well as vehicles, bicycles, and pedestrians.
- (22) *Tracking width.* The total width needed by the tires to traverse a curve; it is the distance measured along the curve radius from the outer front tire track to the inner rear tire track as the vehicle traverses around a curve. This width is used to determine the minimum width required for the vehicle turning. Consideration for additional width may be needed for other vehicles, bicycles and pedestrians.
- (23) *Truck Apron.* The traversable portion of the roundabout central island adjacent to the circulatory roadway that may be needed to accommodate the wheel tracking of large vehicles. A truck apron is sometimes provided on the outside of the circulatory roadway, but cannot encroach upon the pedestrian crossing.
- (24) *Weaving Section.* A length of roadway, designed to accommodate two traffic streams merging and diverging within a short distance.
- (25) *Wheelbase.* For single-unit vehicles, the distance from the first axle to the single rear axle or, in the case of a tandem or triple set of rear axles, to the center of the group of rear axles. See Topic 404

62.5 Landscape Architecture

- (1) *“A” Soil Horizon.* Formed below the “O” soil horizon layer, defined in part (9) below, where mineral matter is mixed with decayed organic matter.
- (2) *Classified Landscaped Freeway.* A classified landscaped freeway is a planted section of freeway that meets the criteria established by the California Code of Regulations Outdoor Advertising Regulations, Title 4, Division 6. This designation is used in the control and regulation of outdoor advertising displays.
- (3) *Duff.* A vegetative material that has been collected and removed from the project during clearing and grubbing activities, or chipped or ground up and stockpiled for reapplication to the final slope surface.
- (4) *Highway Planting.* Highway planting addresses safety requirements, complies with environmental commitments, and assists in the visual integration of the transportation facility within the existing natural and built environment. Highway planting provides planting to satisfy legal mandates, environmental mitigation requirements, Memoranda of Understanding or Agreement between the Department and local agencies for aesthetics or erosion control. Highway planting also includes roadside management strategies that improve worker safety by reducing the frequency and duration of worker exposure.

Highway planting required due to the impacts of a roadway construction project must be programmed and funded by the parent roadway project.

Highway planting, funded and maintained by the Department on conventional highways, is limited to planting that provides: safety improvements, erosion control/stormwater pollution prevention, revegetation, and required mitigation planting. Highway planting on freeways, controlled access highways and expressways, funded and maintained by the Department, is limited to areas that meet specific criteria. See Chapter 29 “Landscape Architecture” of the Project Development Procedures Manual (PDPM) for

more detailed information regarding warranted planting.

- (5) *Highway Planting Revegetation.* Highway planting revegetation provides planting as mitigation for native vegetation damaged or removed due to a roadway construction project. Highway planting revegetation may include irrigation systems as appropriate. Highway planting revegetation, required due to the impacts of a roadway construction project, must be programmed and funded by the parent roadway project.
- (6) *Imported Topsoil.* Soil that is delivered onto a project from a commercial source and is fertile, friable soil of loamy character that contains organic matter.
- (7) *Local Topsoil.* Existing soil obtained from the “A” and “O” soil horizons within the project limits, typically during excavation activities.
- (8) *“O” Soil Horizon.* The surface layer consisting of loose and partly decaying organic matter.
- (9) *Park and Ride.* A paved area for parking which provides a connection point for public access to a variety of modal options. See Topic 905.
- (10) *Replacement Highway Planting.* Replacement highway planting replaces vegetation installed by the Department or others, that has been damaged or removed due to transportation project construction. Replacement highway planting may also include irrigation modifications and/or replacement. Replacement highway planting required due to the impacts of a roadway construction project must be programmed in conjunction with and funded from the parent roadway project.
- (11) *Required Mitigation Planting.* Required mitigation planting provides planting and other work necessary to mitigate environmental impacts due to roadway construction. The word “required” indicates that the work is necessary to meet legally required environmental mitigation or permit requirements. Required mitigation planting may be performed within the operational right of way, immediately adjacent to the highway

or at an offsite location as determined by the permit. A planting project for required mitigation due to the impacts of a roadway construction project must be programmed and funded by the parent roadway project.

- (12) *Roadside Rehabilitation.* The primary purpose of this program is to provide for replacement, restoration and rehabilitation of existing roadside elements, including highway planting and irrigation, following damage by weather, acts of nature or deterioration. This program also provides for erosion control to comply with National Pollutant Discharge Elimination System (NPDES) permit requirements, design for safety features, and improvements for roadside appearance and coordination with community character.
- (13) *Safety Roadside Rest Area System.* The safety roadside rest area system is a component of the highway system providing roadside areas where travelers can stop, rest and manage their travel needs. Planned with consideration of alternative stopping opportunities such as truck stops, commercial services, and vista points, the rest area system provides public stopping opportunities where they are most needed, usually between large towns and at entrances to major metropolitan areas. Within the safety roadside rest system, individual rest areas may include vehicle parking, picnic tables, sanitary facilities, telephones, water, tourist information panels, traveler service information facilities and vending machines. See Topic 903.
- (14) *Street Furniture.* Features such as newspaper boxes, bicycle racks, bus shelters, benches, art or drinking fountains that occupy space on or alongside pedestrian sidewalks.
- (15) *Vista Point.* Typically a paved dedicated area beyond the shoulder that permits travelers to stop and view a scenic area. In addition to parking areas, amenities such as trash receptacles, interpretive displays, and in some cases, rest rooms, drinking water and telephones may be provided. See Topic 904.

62.6 Right of Way

- (1) *Acquisition.* The process of obtaining rights of way.
- (2) *Air Rights.* The property rights for the control or specific use of a designated airspace involving a highway.
- (3) *Appraisal.* An expert opinion of the market value of property including damages and special benefits, if any, as of a specified date, resulting from an analysis of facts.
- (4) *Business District (or Central Business District).* The commercial and often the geographic heart of a city, which may be referred to as "downtown." Usually contains retail stores, theatres, entertainment and convention venues, government buildings, and little or no industry because of the high value of land. Historic sections may be referred to as "old town."
- (5) *Condemnation.* The process by which property is acquired for public purposes through legal proceedings under power of eminent domain.
- (6) *Control of Access.* The condition where the right of owners or occupants of abutting land or other persons to access in connection with a highway is fully or partially controlled by public authority.
- (7) *Easement.* A right to use or control the property of another for designated purposes.
- (8) *Eminent Domain.* The power to take private property for public use without the owner's consent upon payment of just compensation.
- (9) *Encroachment.* In terms of exceptions and permits, includes, but is not limited to, any structure, object, or activity of any kind or character which is within the State right of way, but it is not a part of the State facility or serving a transportation need.
- (10) *Inverse Condemnation.* The legal process which may be initiated by a property owner to compel the payment of just compensation, where the property has been taken for or damaged by a public purpose.
- (11) *Negotiation.* The process by which property is sought to be acquired for project purposes through mutual agreement upon the terms for transfer of such property.
- (12) *Partial Acquisition.* The acquisition of a portion of a parcel of property.
- (13) *Relinquishment.* A transfer of the State's right, title, and interest in and to a highway, or portion thereof, to a city or county.
- (14) *Right of Access.* The right of an abutting land owner for entrance to or exit from a public road.
- (15) *Severance Damages.* Loss in value of the remainder of a parcel which may result from a partial taking of real property and/or from the project.
- (16) *Vacation.* The reversion of title to the owner of the underlying fee where an easement for highway purposes is no longer needed.

62.7 Pavement

The following list of definitions includes terminologies that are commonly used in California as well as selected terms from the "AASHTO Guide for the Design of Pavement Structures" which may be used by FHWA, local agencies, consultants, etc. in pavement engineering reports and research publications.

- (1) *Asphalt Concrete.* See Hot Mix Asphalt (HMA).
- (2) *Asphalt Rubber.* A blend of asphalt binder, reclaimed tire rubber, and certain additives in which the rubber component is at least 15 percent by weight of the total blend and has reacted in the hot asphalt binder sufficiently to cause swelling of the rubber particles.
- (3) *Asphalt Treated Permeable Base (ATPB).* A highly permeable open-graded mixture of crushed coarse aggregate and asphalt binder placed as the base layer to assure adequate drainage of the structural section, as well as structural support.
- (4) *Base.* A layer of selected, processed, and/or treated aggregate material that is placed immediately below the surface course. It

- provides additional load distribution and contributes to drainage and frost resistance.
- (5) *Basement Soil/Material*. See Subgrade.
- (6) *Borrow*. Natural soil obtained from sources outside the roadway prism to make up a deficiency in excavation quantities.
- (7) *California R-Value*. A measure of resistance to deformation of the soils under saturated conditions and traffic loading as determined by the stabilometer test (CT301). The California R-value, also referred to as R-value, measures the supporting strength of the subgrade and subsequent layers used in the pavement structure. For additional information, see Topic 614.
- (8) *Capital Preventive Maintenance*. Typically, Capital Preventive Maintenance (CAPM) consists of work performed to preserve the existing pavement structure utilizing strategies that preserve or extend pavement service life. The CAPM program is divided into pavement preservation and pavement rehabilitation. For further discussion see Topic 603.
- (9) *Cement Treated Permeable Base (CTPB)*. A highly permeable open-graded mixture of coarse aggregate, portland cement, and water placed as the base layer to provide adequate drainage of the structural section, as well as structural support.
- (10) *Composite Pavement*. These are pavements comprised of both rigid and flexible layers. Currently, for purposes of the procedures in this manual, only flexible over rigid composite pavements are considered composite pavements.
- (11) *Crack*. Separation of the pavement material due to thermal and moisture variations, consolidation, vehicular loading, or reflections from an underlying pavement joint or separation.
- (12) *Crack, Seat, and Overlay (CSO)*. A rehabilitation strategy for rigid pavements. CSO practice requires the contractor to crack and seat the rigid pavement slabs, and place a flexible overlay with a pavement reinforcing fabric (PRF) interlayer.
- (13) *Crumb Rubber Modifier (CRM)*. Scrap rubber produced from scrap tire rubber and other components, if required, and processed for use in wet or dry process modification of asphalt paving.
- (14) *Deflection*. The downward vertical movement of a pavement surface due to the application of a load to the surface.
- (15) *Dense Graded Asphalt Concrete (DGAC)*. See Hot Mix Asphalt (HMA).
- (16) *Depression*. Localized low areas of limited size that may or may not be accompanied by cracking.
- (17) *Dowel Bar*. A load transfer device in a rigid slab usually consisting of a plain round steel bar.
- (18) *Edge Drain System*. A drainage system, consisting of a slotted plastic collector pipe encapsulated in treated permeable material and a filter fabric barrier, with unslotted plastic pipe vents, outlets, and cleanouts, designed to drain both rigid and flexible pavement structures.
- (19) *Embankment*. A prism of earth that is constructed from excavated or borrowed natural soil and/or rock, extending from original ground to the grading plane, and designed to provide a stable support for the pavement structure.
- (20) *Equivalent Single Axle Loads (ESAL's)*. The number of 18-kip standard single axle load repetitions that would have the same damage effect to the pavement as an axle of a specified magnitude and configuration. See Index 613.3 for additional information.
- (21) *Flexible Pavement*. Pavements engineered to transmit and distribute vehicle loads to the underlying layers. The highest quality layer is the surface course (generally asphalt binder mixes) which may or may not incorporate underlying layers of base and subbase. These types of pavements are called "flexible" because the total pavement structure bends or flexes to accommodate deflection bending under vehicle loads. For further discussion, see Chapter 630.

- (22) *Grading Plane*. The surface of the basement material upon which the lowest layer of subbase, base, pavement surfacing, or other specified layer, is placed.
- (23) *Gravel Factor (G_f)*. Refers to the relative strength of a given material compared to a standard gravel subbase material. The cohesiometer values were used to establish the G_f currently used by Caltrans.
- (24) *Hot Mix Asphalt (HMA)*. Formerly known as asphalt concrete (AC), HMA is a graded asphalt concrete mixture (aggregate and asphalt binder) containing a small percentage of voids which is used primarily as a surface course to provide the structural strength needed to distribute loads to underlying layers of the pavement structure.
- (25) *Hot Recycled Asphalt (HRA)*. The use of reclaimed flexible pavement which is combined with virgin aggregates, asphalt, and sometimes rejuvenating agents at a central hot-mix plant and placed in the pavement structure in lieu of using all new materials.
- (26) *Joint Seals*. Pourable, extrudable or premolded materials that are placed primarily in transverse and longitudinal joints in concrete pavement to deter the entry of water and incompressible materials (such as sand that is broadcast in freeze-thaw areas to improve skid resistance).
- (27) *Lean Concrete Base*. Mixture of aggregate, portland cement, water, and optional admixtures, primarily used as a base for portland cement concrete pavement.
- (28) *Longitudinal Joint*. A joint normally placed between roadway lanes in rigid pavements to control longitudinal cracking; and the joint between the traveled way and the shoulder.
- (29) *Maintenance*. The preservation of the entire roadway, including pavement structure, shoulders, roadsides, structures, and such traffic control devices as are necessary for its safe and efficient utilization.
- (30) *Open Graded Asphalt Concrete (OGAC)*. See Open Graded Friction Course (OGFC).
- (31) *Open Graded Friction Course (OGFC)*. Formerly known as open graded asphalt concrete (OGAC), OGFC is a wearing course mix consisting of asphalt binder and aggregate with relatively uniform grading and little or no fine aggregate and mineral filler. OGFC is designed to have a large number of void spaces in the compacted mix as compared to hot mix asphalt. For further discussion, see Topic 631.
- (32) *Overlay*. An overlay is a layer, usually hot mix asphalt, placed on existing flexible or rigid pavement to restore ride quality, to increase structural strength (load carrying capacity), and to extend the service life.
- (33) *Pavement*. The planned, engineered system of layers of specified materials (typically consisting of surface course, base, and subbase) placed over the subgrade soil to support the cumulative vehicle loading anticipated during the design life of the pavement. The pavement is also referred to as the pavement structure and has been referred to as pavement structural section.
- (34) *Pavement Design Life*. Also referred to as performance period, pavement design life is the period of time that a newly constructed or rehabilitated pavement is engineered to perform before reaching a condition that requires CAPM, (see Index 603.4). The selected pavement design life varies depending on the characteristics of the highway facility, the objective of the project, and projected vehicle volume and loading.
- (35) *Pavement Drainage System*. A drainage system used for both asphalt and rigid pavements consisting of a treated permeable base layer and a collector system which includes a slotted plastic pipe encapsulated in treated permeable material and a filter fabric barrier with unslotted plastic pipe as vents, outlets and cleanouts to rapidly drain the pavement structure. For further discussion, see Chapter 650.
- (36) *Pavement Preservation*. Work done, either by contract or by State forces to preserve the ride quality, safety characteristics, functional serviceability and structural integrity of

- roadway facilities on the State highway system. For further discussion, see Topic 603.
- (37) *Pavement Service Life.* Is the actual period of time that a newly constructed or rehabilitated pavement structure performs satisfactorily before reaching its terminal serviceability or a condition that requires major rehabilitation or reconstruction. Because of the many independent variables involved, pavement service life may be considerably longer or shorter than the design life of the pavement. For further discussion, see Topic 612.
- (38) *Pavement Structure.* See Pavement.
- (39) *Pumping.* The ejection of base material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab under vehicular traffic loading. This phenomena is especially pronounced with saturated structural sections.
- (40) *Raveling.* Progressive disintegration of the surface course on asphalt concrete pavement by the dislodgement of aggregate particles and binder.
- (41) *Rehabilitation.* Work undertaken to extend the service life of an existing facility. This includes placement of additional surfacing and/or other work necessary to return an existing roadway, including shoulders, to a condition of structural or functional adequacy, for the specified service life. This might include the partial or complete removal and replacement of portions of the pavement structure. Rehabilitation is divided into pavement rehabilitation activities and roadway rehabilitation activities (see Indexes 603.3 and 603.4).
- (42) *Resurfacing.* A supplemental surface layer or replacement layer placed on an existing pavement to restore its riding qualities and/or to increase its structural (load carrying) strength.
- (43) *Rigid Pavement.* Pavement engineered with a rigid surface course (typically Portland cement concrete or a variety of specialty cement mixes for rapid strength concretes) which may incorporate underlying layers of stabilized or unstabilized base or subbase materials. These types of pavements rely on the substantially higher stiffness of the rigid slab to distribute the vehicle loads over a relatively wide area of underlying layers and the subgrade. Some rigid slabs have reinforcing steel to help resist cracking due to temperature changes and repetitive loading.
- (44) *Roadbed.* The roadbed is that area between the intersection of the upper surface of the roadway and the side slopes or curb lines. The roadbed rises in elevation as each increment or layer of subbase, base or surface course is placed. Where the medians are so wide as to include areas of undisturbed land, a divided highway is considered as including two separate roadbeds.
- (45) *Asphalt Rubber Binder.* A blend of asphalt binder modified with crumb rubber modifier (CRM) that may include less than 15 percent CRM by mass.
- (46) *Rubberized Hot Mix Asphalt (RHMA).* Formerly known as rubberized asphalt concrete (RAC). RHMA is a material produced for hot mix applications by mixing either asphalt rubber or asphalt rubber binder with graded aggregate. RHMA may be gap- (RHMA-G) or open- (RHMA-O) graded.
- (47) *R-value.* See California R-Value.
- (48) *Serviceability.* The ability at time of observation of a pavement to serve vehicular traffic (automobiles and trucks) which use the facility. The primary measure of serviceability is the Present Serviceability Index (PSI), which ranges from 0 (impossible road) to 5 (perfect road).
- (49) *Settlement.* Localized vertical displacement of the pavement structure due to slippage or consolidation of the underlying foundation, often resulting in pavement deterioration, cracking and poor ride quality.
- (50) *Structural Section.* See Pavement Structure.
- (51) *Structural Section Drainage System.* See Pavement Drainage System.

(52) *Subbase.* Unbound aggregate or granular material that is placed on the subgrade as a foundation or working platform for the base. It functions primarily as structural support, but it can also minimize the intrusion of fines from the subgrade into the pavement structure, improve drainage, and minimize frost action damage.

(53) *Subgrade.* Also referred to as basement soil, it is the portion of the roadbed consisting of native or treated soil on which pavement surface course, base, subbase, or a layer of any other material is placed.

(54) *Surface Course.* One or more uppermost layers of the pavement structure engineered to carry and distribute vehicle loads. The surface course typically consists of a weather-resistant flexible or rigid layer, which provides characteristics such as friction, smoothness, resistance to vehicle loads, and drainage. In addition, the surface course minimizes infiltration of surface water into the underlying base, subbase and subgrade. A surface course may be composed of a single layer with one or multiple lifts, or multiple layers of differing materials.

(55) *Tie Bars.* Deformed reinforcing bars placed at intervals that hold rigid pavement slabs in adjoining lanes and exterior lane-to-shoulder joints together and prevent differential vertical and lateral movement.

62.8 Highway Operations

(1) *Annual Average Daily Traffic.* The average 24-hour volume, being the total number during a stated period divided by the number of days in that period. Unless otherwise stated, the period is a year. The term is commonly abbreviated as ADT or AADT.

(2) *Delay.* The time lost while road users are impeded by some element over which the user has no control.

(3) *Density.* The number of vehicles per mile on the traveled way at a given instant.

(4) *Design Vehicles.* See Topic 404.

(5) *Design Volume.* A volume determined for use in design, representing traffic expected to use

the highway. Unless otherwise stated, it is an hourly volume.

(6) *Diverging.* The dividing of a single stream of traffic into separate streams.

(7) *Headway.* The time in seconds between consecutive vehicles moving past a point in a given lane, measured front to front.

(8) *Level of Service.* A rating using qualitative measures that characterize operational conditions within a traffic stream and their perception by users.

(9) *Managed Lanes.* Lanes that are proactively managed in response to changing operating conditions in efforts to achieve improved efficiency and performance. Typically employed on highways with increasing recurrent traffic congestion and limited resources.

(a) High-Occupancy Vehicle (HOV) Lanes--An exclusive lane for vehicles carrying the posted number of minimum occupants or carpools, either part time or full time.

(b) High Occupancy Toll (HOT) Lanes--An HOV lane that allows vehicles qualified as carpools to use the facility without a fee, while vehicles containing less than the required number of occupants to pay a toll. Tolls may change based on real time conditions (dynamic) or according to a schedule (static).

(c) Express Toll Lanes--Facilities in which all users are required to pay a toll, although HOVs may be offered a discount. Tolls may be dynamic or static.

(10) *Merging.* The converging of separate streams of traffic into a single stream.

(11) *Running Time.* The time the vehicle is in motion.

(12) *Spacing.* The distance between consecutive vehicles in a given lane, measured front to front.

(13) *Speed.*

(a) Design Speed--A speed selected to establish specific minimum geometric

**Table 82.1A
Mandatory Standards (Cont.)**

CHAPTER 700	MISCELLANEOUS STANDARDS		Bikeways ^{(1), (2)}
Topic 701	Fences		1003.1 Class I Bikeway in Freeway Medians ^{(1), (2)}
Index 701.2	Fences on Freeways and Expressways ⁽¹⁾		1003.1 Class I Bikeway Design Speeds ^{(1), (2)}
			1003.1 Stopping Sight Distance ⁽²⁾
CHAPTER 900	LANDSCAPE ARCHITECTURE		1003.1 Bikeway Shoulder Slope ⁽¹⁾⁽²⁾
Topic 902	Planting Guidelines		1003.1 Obstacle Posts or Bollards in Bicycle Paths ⁽²⁾
Table 902.3	Large Tree Setback Requirements on Conventional Highways – Median with Curb ⁽¹⁾	CHAPTER 1100	HIGHWAY TRAFFIC NOISE ABATEMENT
902.3	Large Tree Setback Requirements on Conventional Highways – Median with Barrier ⁽¹⁾	Topic 1102	Design Criteria
902.3	The Planting of Trees From Manholes on Conventional Highway Medians ⁽¹⁾	Index 1102.2	Horizontal Clearance to Noise Barrier
902.3	The Planting of Trees From the Logitudinal End of Conventional Highway Medians ⁽¹⁾	1102.2	Noise Barrier on Safety Shape Concrete Barrier ⁽¹⁾
Topic 903	Safety Roadside Rest Area Design Standards and Guidelines		
Index 903.5	Rest Area Ramp Design		
Topic 904	Vista Point Standards and Guidelines		
Index 904.3	Vista Point Ramp Design		
CHAPTER 1000	BICYCLE TRANSPORTATION DESIGN		
Topic 1003	Design Criteria		
Index 1003.1	Class I Bikeway Widths ^{(1), (2)}		
1003.1	Class I Bikeway Shoulder Width ^{(1), (2)}		
1003.1	Class I Bikeway Horizontal Clearance ^{(1), (2)}		
1003.1	Class I Bikeway Structure Width ^{(1), (2)}		
1003.1	Class I Bikeway Vertical Clearance ^{(1), (2)}		
1003.1	Class I Bikeway Minimum Separation From Edge of Traveled Way ^{(1), (2)}		
1003.1	Physical Barriers Adjacent to Class I		

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| <p>(1) Caltrans-only Mandatory Standard.</p> <p>(2) Authority to approve deviations from this Mandatory Standard is delegated to the District Director.</p> <p>(3) Authority to approve deviations from this Mandatory Standard is delegated to the State Pavement Engineer.</p> |
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**Table 82.1B
Advisory Standards**

CHAPTER 100	BASIC DESIGN POLICIES	202.7	Superelevation on City Streets and County Roads
Topic 101	Design Speed	Topic 203	Horizontal Alignment
Index 101.1	Selection of Design Speed – Local Facilities	Index 203.1	Horizontal Alignment – Local Facilities
101.1	Selection of Design Speed – Local Facilities – with Connections to State Facilities	203.3	Alignment Consistency and Design Speed
101.2	Design Speed Standards	203.5	Compound Curves
Topic 104	Control of Access	203.5	Compound Curves on One-Way Roads
Index 104.5	Relation of Access Opening to Median Opening	203.6	Reversing Curves – Transition Length
Topic 105	Pedestrian Facilities	203.6	Reversing Curves – Transition Rate
Index 105.2	Minimum Sidewalk Width – Next to a Building	Topic 204	Grade
105.2	Minimum Sidewalk Width – Not Next to a Building	Index 204.1	Standards for Grade – Local Facilities
105.5	New Construction, Two Curb Ramp Design	204.3	Standards for Grade
Topic 107	Roadside Installations	204.3	Ramp Grades
Index 107.1	Standards for Roadway Connections	204.4	Vertical Curves – 2 Percent and Greater
107.1	Number of Exits and Entrances Allowed at Roadway Connections	204.4	Vertical Curves – Less Than 2 Percent
CHAPTER 200	GEOMETRIC DESIGN AND STRUCTURE STANDARDS	204.5	Decision Sight Distance at Climbing Lane Drops
Topic 201	Sight Distance	204.6	Horizontal and Vertical Curves Consistency in Mountainous or Rolling Terrain
Index 201.3	Stopping Sight Distance on Sustained Grades	Topic 205	Road Connections and Driveways
201.7	Decision Sight Distance	Index 205.1	Access Opening Spacing on Expressways
Topic 202	Superelevation	205.1	Access Opening Spacing on Expressways – Location
Index 202.2	Superelevation on Same Plane for Rural Two-lane Roads	Topic 206	Pavement Transitions
202.2	Superelevation on Class II and III Bikeways	Index 206.3	Lane Drop Transitions
202.5	Superelevation Transition	206.3	Lane Width Reductions
202.5	Superelevation Runoff	Topic 208	Bridges, Grade Separation Structures, and Structure Approach Embankment
202.5	Superelevation in Restrictive Situations	Index 208.3	Decking of Bridge Medians
202.6	Superelevation of Compound Curves	208.6	Minimum Height of Pedestrian Undercrossings
		208.6	Class I Bikeways Exclusive Use

**Table 82.1B
Advisory Standards (Cont.)**

	208.10	Protective Screening on Overcrossings		309.1	Safety Shaped Barriers at Retaining, Pier, or Abutment Walls
	208.10	Bicycle Railing Locations		309.1	High Speed Rail Clearance
Topic 210		Earth Retaining Systems		309.5	Structures Across or Adjacent to Railroads – Vertical Clearance
Index	210.6	Cable Railing			
CHAPTER 300		GEOMETRIC CROSS SECTION	Topic 310		Frontage Roads
			Index	310.2	Outer Separation – Urban and Mountainous Areas
Topic 301		Traveled Way Standards		310.2	Outer Separation – Rural Areas
Index	301.2	Class II Bikeway Lane Width	CHAPTER 400		INTERSECTIONS AT GRADE
	301.3	Algebraic Differences of Cross Slopes at Various Locations	Topic 403		Principles of Channelization
Topic 303		Curbs, Dikes, and Side Gutters	Index	403.3	Angle of Intersection
	303.1	Use of Curb with Posted Speeds of 40 mph and Greater		403.6	Optional Right-Turn Lanes
	303.3	Dike Selection		403.6	Right-Turn-Only Lane and Bike Lane
	303.4	Bulbout Design	Topic 404		Design Vehicles and Related Definitions
	303.4	Bulbouts at Mid-block locations	Index	404.4	STAA Design Vehicles on the National Network and on Terminal Access Routes
	303.4	Curb Face Setback at Bulbouts		404.4	California Legal Design Vehicle Accommodation
Topic 304		Side Slopes		404.4	45-Foot Bus and Motorhome Design Vehicle
Index	304.1	Side Slopes 4:1 or Flatter	Topic 405		Intersection Design Standards
	304.1	18 ft Minimum Catch Distance	Index	405.1	Corner Sight Distance at Unsignalized Public Road Intersections
Topic 305		Median Standards		405.1	Decision Sight Distance at Intersections
Index	305.1	Median Width Freeways and Expressways – Urban		405.4	Pedestrian Refuge by Area Place Type
	305.1	Median Width Freeways and Expressways – Rural		405.5	Emergency Openings and Sight Distance
	305.1	Median Width Conventional Highways – Urban and Rural Main Streets		405.5	Median Opening Locations
	305.1	Median Width Conventional Highways – Climbing or Passing Lanes		405.10	Entry Speeds – Single and Multilane Roundabouts
	305.2	Median Cross Slopes	CHAPTER 500		TRAFFIC INTERCHANGES
Topic 308		Cross Sections for Roads Under Other Jurisdictions	Topic 504		Interchange Design Standards
Index	308.1	Cross Section Standards for City Streets and County Roads without Connection to State Facilities	Index	504.2	Ramp Entrance and Exit Standards
Topic 309		Clearances			
Index	309.1	Clear Recovery Zone			
	309.1	Horizontal Clearance			

Table 82.1B
Advisory Standards (Cont.)

504.2	Collector-Distributor Deceleration Lane and "DL" Distance	504.3	Two-lane Exit Ramps and Auxiliary Lanes
504.2	Paved Width at Gore	504.3	Distance Between Successive On-ramps
504.2	Contrasting Surface Treatment	504.3	Distance Between Successive Exits
504.2	Auxiliary Lanes	504.4	Freeway-to-freeway Connections Design Speed
504.2	Freeway Exit Nose Design Speed	504.4	Profile Grades on Freeway-to-freeway Connectors
504.2	Decision Sight Distance at Exits and Branch Connections	504.4	Single-lane Freeway-to-freeway Connector Design
504.2	Design Speed and Alignment Consistency at Inlet Nose	504.4	Single-lane Connector Widening for Passing
504.2	Freeway Ramp Profile Grades	504.4	Volumes Requiring Branch Connectors
504.2	Differences in Pavement Cross Slopes at Freeway Entrances and Exits	504.4	Merging Branch Connector Design
504.2	Vertical Curves Beyond Freeway Exit Nose	504.4	Diverging Branch Connector Design
504.2	Crest Vertical Curves at Freeway Exit Terminal	504.4	Merging Branch Connector Auxiliary Lanes
504.2	Sag Vertical Curves at Freeway Exit Terminal	504.4	Diverging Branch Connector Auxiliary Lanes
504.2	Ascending Entrance Ramps with Sustained Upgrades	504.4	Freeway-to-freeway Connector Lane Drop Taper
504.3	Ramp Terminus Design Speed	504.6	Mainline Lane Reduction at Interchanges
504.3	Ramp Lane Drop Taper At 6-foot Separation Point	504.8	Access Control at Ramp Terminal
504.3	Ramp Lane Drop Location		
504.3	Metered Single-Lane Entrance Ramps Truck Volumes and Grades		
504.3	Metered Multi-Lane Entrance Ramps Lane Drop		
504.3	Metered Multi-Lane Entrance Truck Volumes and Sustained Grades		
504.3	Ramp Terminals and Grade		
504.3	Ramp Terminals and Sight Distance		
504.3	Free Right-Turns at Ramp Terminals		
504.3	Distance between Ramp Intersection and Local Road Intersection		
504.3	Entrance Ramp Lane Drop		
504.3	Single-Lane Ramp Widening for Passing		
504.3	Two-lane Exit Ramps		

CHAPTER 610 PAVEMENT ENGINEERING CONSIDERATIONS

Topic 612 Pavement Design Life

Index 612.6 Traffic Loading for Temporary Pavements and Detours

CHAPTER 620 RIGID PAVEMENT

Topic 625 Engineering Procedures for Pavement and Roadway Rehabilitation

Index 625.1 Repair of Existing Pavement Distresses

Table 82.1B

Advisory Standards (Cont.)

CHAPTER 630 FLEXIBLE PAVEMENT

Topic 635 Engineering Procedures for Flexible Pavement and Roadway Rehabilitation

Index 635.1 Repair of Existing Pavement Distresses

CHAPTER 640 COMPOSITE PAVEMENTS

Topic 645 Engineering Procedures for Pavement and Roadway Rehabilitation

Index 645.1 Repair of Existing Pavement Distresses

CHAPTER 700 MISCELLANEOUS STANDARDS

Topic 701 Fences

Index 701.2 Fences on Freeways and Expressways

CHAPTER 900 LANDSCAPE ARCHITECTURE

Topic 902 Planting Guidance

Index 902.2 Clear Recovery Zone Planting of Large Trees on Freeways and Expressways, Including Interchanges

902.2 Minimum Tree Setback

Table 902.3 Large Tree Setback Requirements on Conventional Highways - Roadside

Topic 904 Vista Point Standards and Guidelines

Index 904.3 Road Connections to Vista Points

CHAPTER 1000 BICYCLE TRANSPORTATION DESIGN

Topic 1003 Bikeway Design Criteria

Index 1003.1 Class I Bikeway Horizontal Clearance

1003.1 Class I Bikeway in State Highway or Local Road Medians

Table 82.1C
Decision Requiring Other Approvals

CHAPTER 100	BASIC DESIGN POLICIES	Topic 208.10	Bridge Barriers and Railing
		Index 208.10	Barrier Separation and Bridge Rail Selection
Topic 103	Design Designation	208.10	Concrete Barrier Type 80
Index 103.2	Design Period	208.10	Concrete Barrier Type 80SW
Topic 108	Coordination With Other Agencies	208.11	Deviations from Foundation and Embankment Recommendations
Index 108.2	Transit Loading Facilities – Location	210.4	Cost Reduction Incentive Proposals
108.2	Transit Loading Facilities - ADA		
108.3	Rail Crossings*		
108.3	Parallel Rail Facilities*		
108.5	Bus Rapid Transit – Location and ADA		
108.7	Coordination With the FHWA - Approvals		
Topic 110	Special Considerations		
Index 110.1	Overload Category		
110.8	Safety Review Items and Employee Exposure		
110.10	Proprietary Items		
110.10	Proprietary Items – On Structure		
110.10	Proprietary Items – National Highway System		
Topic 111	Material Sites and Disposal Sites		
Index 111.1	Mandatory Material Sites on Federal-aid Projects		
111.6	Mandatory Material Sites and Disposal Sites on Federal-aid Projects		
Topic 116	Bicyclists and Pedestrians on Freeway		
Index 116	Bicycles and Pedestrians on Freeways		
CHAPTER 200	GEOMETRIC DESIGN AND STRUCTURE STANDARDS		
Topic 204	Grade		
Index 204.8	Grade Line of Structures – Temporary Vertical Clearances		
Topic 205	Road Connections and Driveways		
Index 205.1	Conversion of a Private Opening		
		CHAPTER 300	GEOMETRIC CROSS SECTION
		Topic 303	Curbs, Dikes, and Side Gutters
		Index 303.4	Busbulbs
		Topic 304	Side Slopes
		Index 304.1	Side Slopes – Erosion Control
		304.1	Side Slopes – Structural Integrity
		309.2	Vertical Clearance on National Highway System
		309.2	Vertical Clearance Above Railroad Facilities
		309.5	Horizontal and Vertical Clearances at Railroad Structures
		CHAPTER 500	TRAFFIC INTERCHANGES
		Topic 502	Interchange Types
		Index 502.2	Single Point Interchange Interchanges
		502.2	Other Types of Interchanges
		Topic 503	Interchange Procedure
		Index 503.2	Interchange Geometrics
		Topic 504	Interchange Design Standards
		Index 504.3	HOV Preferential Lane
		504.3	Modification to Existing HOV Preferential Lanes
		504.3	Enforcement Areas and Maintenance Pullouts – Required Enforcement Area
		504.3	Enforcement Areas and Maintenance Pullouts – Removal

* Authority to approve deviations from this “Decision Requirement” is delegated to the District Director.

Table 82.1C
Decision Requiring Other Approvals (Cont.)

504.3	Enforcement Areas and Maintenance Pullouts - Length	CHAPTER 800	HIGHWAY DRAINAGE DESIGN
504.6	Mainline Lane Reduction	Topic 805	Preliminary Plans
CHAPTER 600	PAVEMENT ENGINEERING	Index 805.1	Requires FHWA Approval
Topic 604	Roles and Responsibilities for Pavement Engineering	805.2	Bridge Preliminary Report
Index 604.2	Standard Plans	805.4	Unusual Hydraulic Structures
604.2	Supplemental District Standards	805.5	Levees and Dams Formed by Highway Fills
Topic 606	Research and Special Designs	805.6	Geotechnical
Index 606.1	Research and Experimentation – Pilot Projects	Topic 808	Selected Computer Programs
606.1	Research and Experimentation – Special Designs	Index 808.1	Table 808.1
CHAPTER 610	PAVEMENT ENGINEERING CONSIDERATIONS	CHAPTER 820	CROSS DRAINAGE
Topic 614	Other Considerations	Topic 829	Other Considerations
Index 614.6	Compaction	Index 829.9	Dams
CHAPTER 620	RIGID PAVEMENT	CHAPTER 830	TRANSPORTATION FACILITY DRAINAGE
Topic 626	Other Considerations	Topic 837	Inlet Design
Index 626.2	Shoulder – Widened Slab	Index 837.2	Inlet Types
CHAPTER 700	MISCELLANEOUS STANDARDS	CHAPTER 850	PHYSICAL STANDARDS
Topic 701	Fences	Topic 853	Pipe Liners and Linings for Culvert Rehabilitation
Index 701.1	Fence Type and Location	Index 853.4	Alternative Pipe Liner Materials
701.2	Locked Gates - Maintenance Force Use	CHAPTER 870	CHANNEL AND SHORE PROTECTION – EROSION CONTROL
701.2	Locked Gates - Used by Utility Companies*	Topic 872	Planning and Location Studies
701.2	Locked Gates - Used by Other Public Agencies or by Non-Utility Entities – FHWA Approval Required on Interstates	Index 872.3	Site Consideration
Topic 706	Roadside Treatment	Topic 873	Design Concepts
Index 706.2	Vegetation Control	Index 873.1	Introduction
		873.3	Armor Protection
		CHAPTER 900	LANDSCAPE ARCHITECTURE
		Topic 901	General
		Index 901.1	Landscape Architecture Program - Approvals

* Authority to approve deviations from this “Decision Requirement” is delegated to the District Director.

Table 82.1C Decision Requiring Other Approvals (Cont.)

Topic 902	Planting Guidelines
Index 902.3	Plant Selection, Setback and Spacing
Table 902.3	Large Tree Setback Requirements on Conventional Highway Medians in Main Street Context
Table 902.3	Planting of Large Trees on Conventional Highway Medians – With Barrier and Posted Speed Greater Than 45mph
Topic 903	Safety Roadside Rest Areas Standards and Guidelines
Index 903.1	Deviation From Minimum Standard
903.6	Wastewater Disposal
Topic 904	Vista Point Standards and Guidelines
Index 904.1	Site Selection
904.3	Sanitary Facilities
Topic 905	Park and Ride Standards and Guidelines
Index 905.1	Site Selection
CHAPTER 1000	BICYCLE TRANSPORTATION DESIGN
Topic 1003	Miscellaneous Criteria
Index 1003.5	Bicycle Path at Railroad Crossings
CHAPTER 1100	HIGHWAY TRAFFIC NOISE ABATEMENT
Topic 1101	General Requirements
Index 1101.2	Objective – Extraordinary Abatement

* Authority to approve deviations from this “Decision Requirement” is delegated to the District Director.

It is recognized that in many cases provisions for traffic control will be dependent on the way the contractor chooses to execute the project, and that the designer may have to make some assumptions as to the staging or sequence of the contractor's operations in order to develop definite temporary traffic control plans. However, safety of the public and the workers as well as public convenience demand that designers give careful consideration to the plans for handling all traffic even though a different plan may be followed ultimately. It is simpler from a contract administration standpoint to change a plan than to add one where none existed. The special provisions should specify that the contractor may develop alternate traffic control plans if they are as sound or better than those provided in the contract PS&E.

See Section 2-30, Traffic, of the Construction Manual for additional factors to be considered in the preparation of traffic control plans.

110.8 Safety Reviews

Formal safety reviews during planning, design and construction have demonstrated that safety-oriented critiques of project plans help to ensure the application of safety standards. An independent team not involved in the design details of the project is generally able to conduct reviews from a fresh perspective. In many cases, this process leads to highly cost-effective modifications that enhance safety for motorists, bicyclists, pedestrians, and highway workers without any material changes in the scope of the project.

(1) *Policy.* During the planning stage all projects must be reviewed by the District Safety Review Committee prior to approval of the appropriate project initiation document (PSR, PSSR, NBSSR, etc.).

During design, each major project with an estimated cost over the Minor A limit must be reviewed by the District Safety Review Committee.

Any project, regardless of cost, requiring a Traffic Control Plan must be reviewed by the District Safety Review Committee. During construction, the detection of the need for safety-related changes is the responsibility of construction personnel, as outlined in the Construction Manual.

Safety concepts that are identified during these safety reviews which directly limit the exposure of employees to vehicular and bicycle traffic shall be incorporated into the project unless deletion is approved by the District Director.

(2) *Procedure.* Each District must have a Safety Review Committee, composed of at least one engineer from the Construction, Design, Maintenance, and Traffic functions and should designate one of the members as chairperson. Committee members should familiarize themselves with current standards and instructions on highway safety so that they can identify items in need of correction.

The Committee should conduct at least two design safety reviews of each major project. The Design Project Engineer has the basic responsibility to notify the committee chairperson when a review is needed. The chairperson should schedule a review and coordinate participation by appropriate committee members.

Reviews, evaluating safety from the perspectives of the motorists, bicyclists, and pedestrians, should include qualitative and/or quantitative safety considerations of such items as:

- Exposure of employees to vehicular and bicycle traffic.
- Traffic control plans.
- Transportation Management Plans.
- Traversability of roadsides.
- Elimination or other appropriate treatment of fixed objects.
- Susceptibility to wrong-way moves.
- Safety of construction and maintenance personnel.
- Sight distance.
- ADA design.
- Guardrail.
- Run off road concerns.
- Superelevation, etc.

- Roadside management and maintenance reduction.
- Access to facilities from off of the freeway.
- Maintenance vehicle pull-out locations.

The objective is to identify all elements where safety improvement may be practical and indicate desirable corrective measures. Reviews should be scheduled when the report or plans are far enough along for a review to be fruitful, but early enough to avoid unnecessary delay in the approval of the report or the completion of PS&E.

A simple report should be prepared on the recommendations made by the Safety Committee and the response by the Design Project Engineer. The reports should be included in the project files.

110.9 Value Analysis

The use of Value Analysis techniques should begin early in the project development process and be applied at various milestones throughout the PS&E stage to reduce life-cycle costs. See the Project Development Procedures Manual for additional information.

110.10 Proprietary Items

The use of proprietary items is discouraged in the interest of promoting competitive bidding. If it is determined that a proprietary item is needed and beneficial to the State, their use must be approved by the District Director or by the Deputy District Director of Design (if such approval authority has been specifically delegated by the District Director). The Division Chief of Engineering Services must approve the use of proprietary items on structures and other design elements under their jurisdiction. The Department's guidelines on how to include proprietary items in contract plans are covered in the Office Engineer's Ready to List and Construction Contract Award Guide (RTL Guide) under "Proprietary Products."

On projects that utilize federal funds, the use of proprietary items requires an additional approval through a Public Interest Finding (PIF). A PIF is approved by the Federal Highway Administration (FHWA) Division Office for "High Profile

Projects" or by the Division of Budgets, California Federal Resources Engineer for Delegated Projects, in accordance with the Stewardship Agreement. Additional information on the PIF process can be found through the Division of Budgets, Office of Federal Resources.

The use of proprietary materials, methods, or products will not be approved unless:

- (a) There is no other known material of equal or better quality that will perform the same function, or
- (b) There are overwhelming reasons for using the material or product in the public's interest, which may or may not include cost savings, or
- (c) It is essential for synchronization with existing highway or adjoining facilities, or
- (d) Such use is on an experimental basis, with a clearly written plan for "follow-up and evaluation."

If the proprietary item is to be used experimentally and there is Federal participation, the request for FHWA approval must be submitted to the Chief, Office of Resolution of Necessity, Encroachment Exceptions, and Resource Conservation in the Division of Design. The request must include a Construction Evaluated Work Plan (CEWP), which indicates specific functional managers, and units, which have been assigned responsibility for objective follow-up, evaluation, and documentation of the effectiveness of the proprietary item.

110.11 Conservation of Materials and Energy

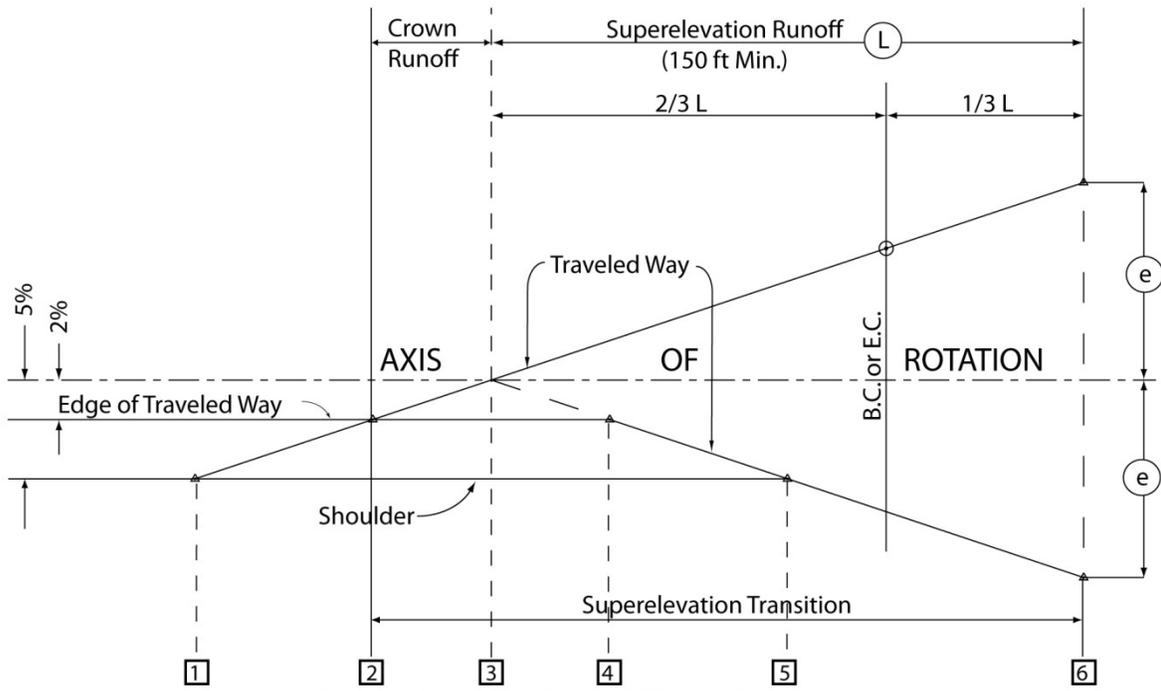
Paving materials such as cement, asphalt, and rock products are becoming more scarce and expensive, and the production processes for these materials consume considerable energy. Increasing evidence of the limitation of nonrenewable resources and increasing worldwide consumption of most of these resources require optimal utilization and careful consideration of alternates such as the substitution of more plentiful or renewable resources and the recycling of existing materials.

- (1) *Rigid Pavement.* The crushing and reuse of old rigid pavement as aggregate in new rigid or flexible pavement does not now appear to

**Figure 202.5A
Superelevation Transition**

Formulas		Explanation of Terms
2-Lane Roads	$L = 2500 e$	(L) = Length of Superelevation Runoff - ft
Multilane Roads & Branch Connections	$L = 150 D e$	(e) = Superelevation rate - ft/ft
Ramps		(D) = Distance from axis of rotation to outside edge of lanes - ft
Multilane	$L = 2500 e$ if possible	
Single Lane	$L = 2000 e$	
MINIMUM L = 150 FT		MAXIMUM L = 510 FT

Adjust computed length to nearest 10 ft. length divisible by 3



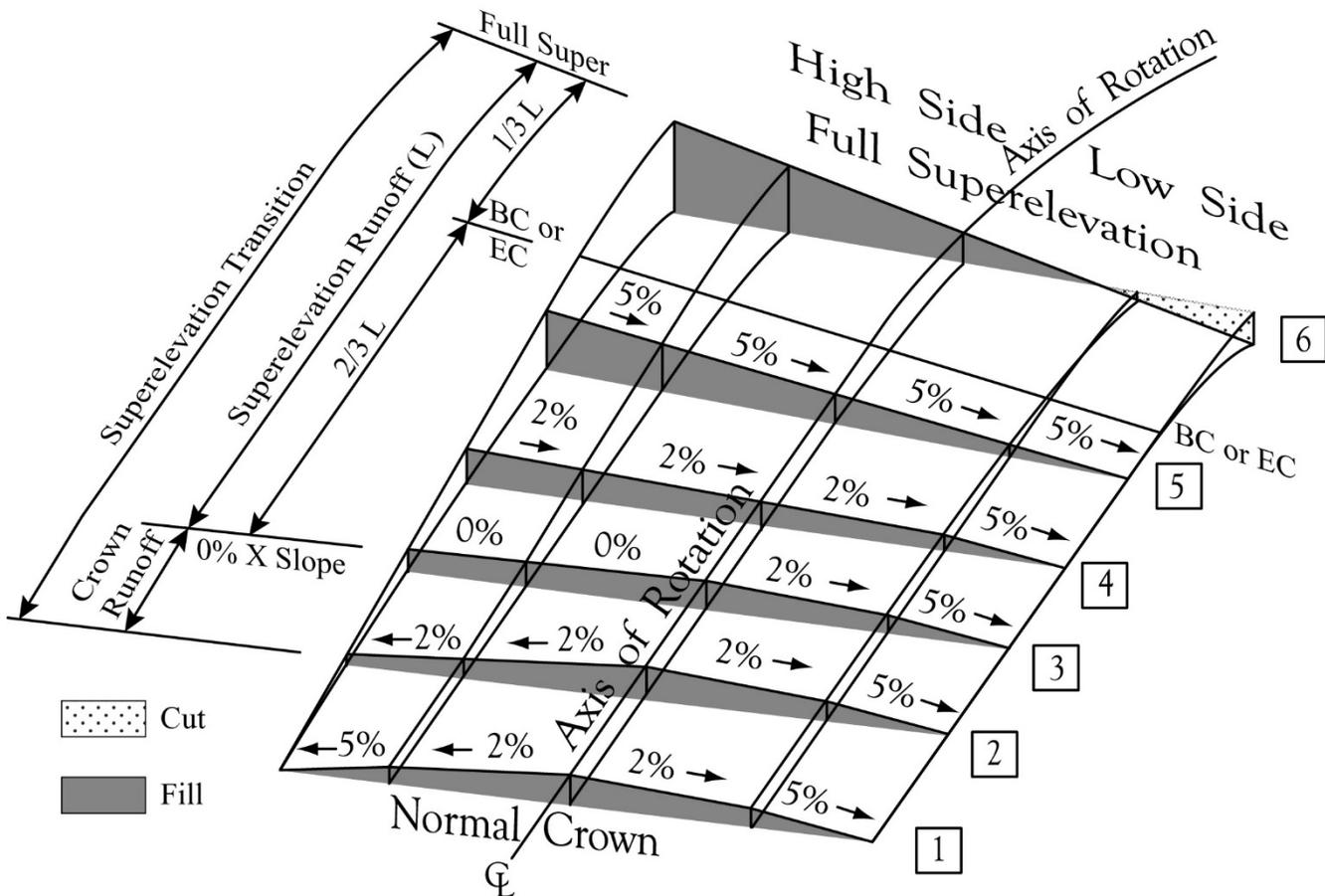
Superelevation Runoff Lengths

Superelevation Rate "e" ft/ft	2-Lane Highways & Multilane Ramps	Single Lane Ramps	Length, L (feet)						
			24 ft	36 ft	48 ft	51 ft	60 ft	63 ft	75 ft
0.02	150	150	150	150	150	150	180	180	240
0.03	150	150	150	180	210	240	270	270	330
0.04	150	150	150	210	300	300	360	390	450
0.05	150	150	180	270	360	390	450	480	510
0.06	150	150	210	330	450	450	510	510	
0.07	180	150	270	390	510	510			
0.08	210	150	300	450					
0.09	240	180	330	480					
0.10	240	210	360	510					
0.11	270	210	390						
0.12	300	240	420						

For widths of "D" not included in table, use formula above.

Figure 202.5B
Superelevation Transition Terms & Definitions

Term	Definition
Crown Runoff 2 ↔ 3	The distance from the station where the high side of the superelevating section surfaces are at a cross slope of 2% to where the high side of the section surfaces reaches a cross slope of 0%.
Superelevation Runoff (L) 3 ↔ 6	The distance from the station where the high side of the superelevating section surfaces are at a cross slope of 0% to the station where the entire cross section is at full superelevation.
Superelevation Transition 2 ↔ 6	The distance from the station where the high side of the superelevating sections are crowned at a cross slope of 2% to the station where the entire cross section is at full superelevation. The Crown Runoff Length plus the Superelevation Runoff Length (L) equals the Superelevation Transition Length.
%L On tangent	The percentage of the superelevation runoff length (L) that is outside of the curve (2/3L). See Index 202.5(2).
%L On curve	The percentage of the superelevation runoff length (L) that is within the curve (1/3L). See Index 202.5(2). The % On Tangent and % On curve values must total 100%.

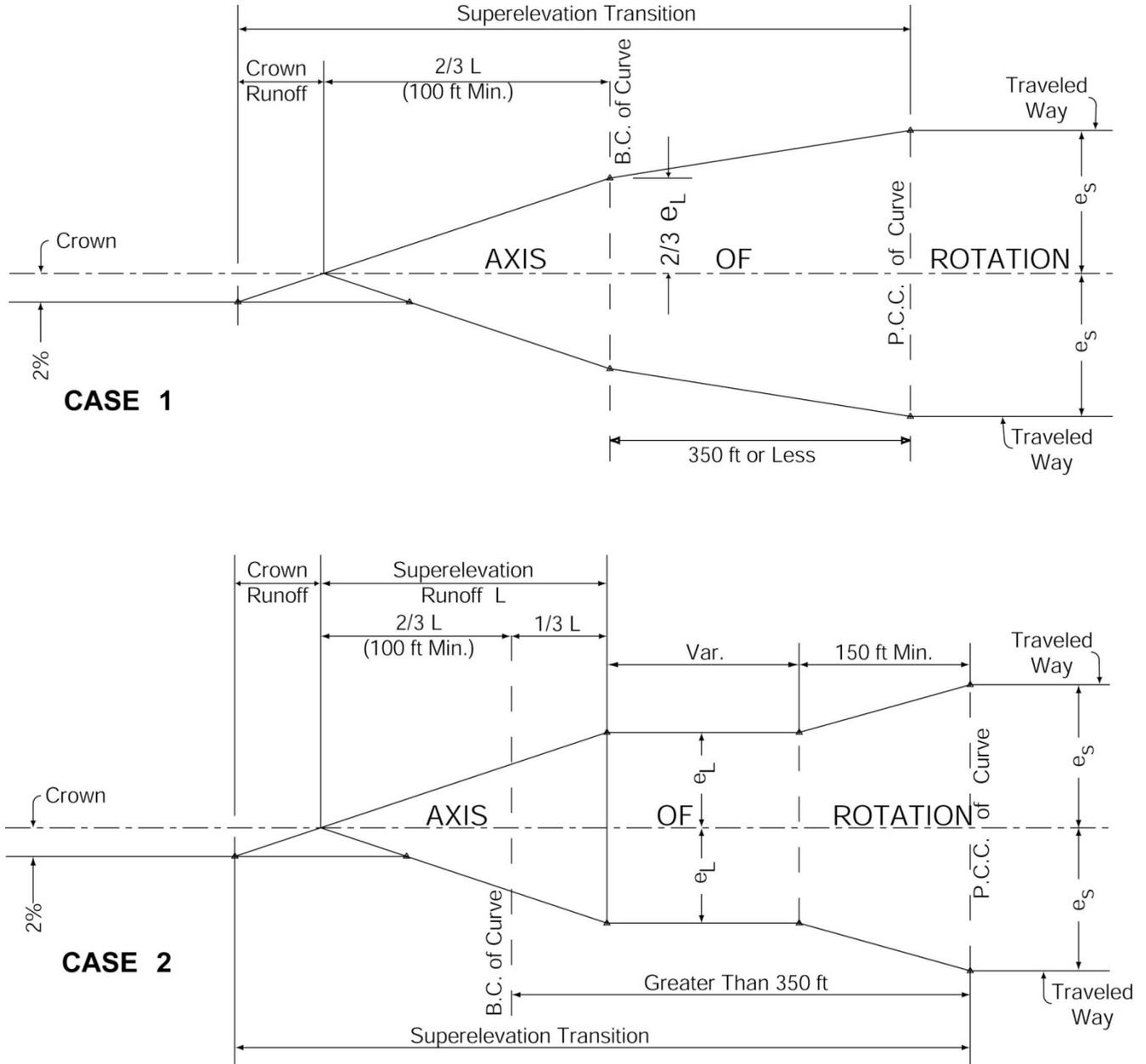


Elements of a Superelevation Transition (Right Curve)

Figure 202.6

Superelevation of Compound Curves

- L = Length of superelevation runoff - ft
- e_s = Superelevation rate for smaller radius curve - ft/ft or percent
- e_L = Superelevation rate for larger radius curves - ft/ft or percent



203.2 Standards for Curvature

Table 203.2 shall be the minimum radius of curve for specific design speeds on highways. This table is based upon speed alone; it does not address the sight distance factor. **If the minimum radii indicated in Table 203.2 does not provide the desired lateral clearance to an obstruction, Figure 201.6 shall govern.**

Every effort should be made to exceed minimum values, and such minimum radii should be used only when the cost or other adverse effects of realizing a higher standard are inconsistent with the benefits. As an aid to designers, Figure 202.2 displays the maximum comfortable speed for various curve radii and superelevation rates. Use of Figure 202.2, in lieu of the above standards must be documented as discussed in Index 82.2.

The recommended minimum radii for freeways are 5,000 feet in rural areas and 3,000 feet in urban areas.

If a glare screen or a median barrier is contemplated, either initially or ultimately, adjustments may be necessary to maintain the required sight distance on curves on divided highways. In such cases, a larger curve radius or a wider median may be required throughout the length of the curve. For design purposes, a planting screen is presumed to be 8 feet wide. See Chapter 7 of the Traffic Manual for glare screen criteria.

**Table 203.2
Standards for Curve Radius**

Design Speed mph	Minimum Radius of Curve (ft)
20	130
30	300
40	550
50	850
60	1,150
70	2,100
80	3,900

203.3 Alignment Consistency

Sudden reductions in alignment standards should be avoided. Where physical restrictions on curve radius cannot be overcome and it becomes necessary to introduce curvature of lower standard than the design speed for the project, the design speed between successive curves should change not more than 10 miles per hour. Introduction of curves with lower design speeds should be avoided at the end of long tangents, steep downgrades, or at other locations where high approach speeds may be anticipated.

The horizontal and vertical alignments should be coordinated such that horizontal curves are not hidden behind crest vertical curves. Sharp horizontal curves should not follow long tangents because some drivers tend to develop higher speeds on the tangent and could over drive the curve.

See “Combination of Horizontal and Vertical Alignment” in Chapter III of AASHTO, A Policy on Geometric Design of Highways and Streets, for further guidance on alignment consistency.

203.4 Curve Length and Central Angle

The minimum curve length for central angles less than 10 degrees should be 800 feet to avoid the appearance of a kink. For central angles larger than 30 minutes, a curve is required without exception. Above a 20,000-foot radius, a parabolic curve may be used. Sight distance or other safety considerations are not to be sacrificed to meet the above requirements.

On 2-lane roads a curve should not exceed a length of one-half mile and should be no shorter than 500 feet.

203.5 Compound Curves

Compound curves should be avoided because drivers who have adjusted to the first curve could over drive the second curve if the second curve has a smaller radius than the first. Exceptions can occur in mountainous terrain or other situations where use of a simple curve would result in excessive cost. Where compound curves are necessary, the shorter radius should be at least two-thirds the longer radius when the shorter radius is 1,000 feet or less. On

one-way roads, the larger radius should follow the smaller radius.

The total arc length of a compound curve should be not less than 500 feet.

203.6 Reversing Curves

When horizontal curves reverse direction the connecting tangents should be long enough to accommodate the standard superelevation runoffs given on Figure 202.5. If this is not possible, the 6 percent per 100 feet rate of change should govern (see Index 202.5(3)). When feasible, a minimum of 400 feet of tangent should be considered.

203.7 Broken Back Curves

A broken back curve consists of two curves in the same direction joined by a short tangent. Broken back curves are unsightly and undesirable.

203.8 Spiral Transition

Spiral transitions are used to transition from a tangent alignment to a circular curve and between circular curves of unequal radius. Spiral transitions may be used whenever the traffic lane width is less than 12 feet, the posted speed is greater than 45 miles per hour, and the superelevation rate exceeds 8 percent. The length of spiral should be the same as the Superelevation Runoff Length shown in Figure 202.5A. In the typical design, full superelevation occurs where the spiral curve meets the circular curve, with crown runoff being handled per Figure 202.5A. For a general discussion of spiral transitions see *AASHTO A Policy on the Geometric Design of Streets and Highways*. When used, spirals transitions should conform to the Clothoid definition.

203.9 Alignment at Bridges

Due to the difficulty in constructing bridges with superelevation rates greater than 10 percent, the curve radii on bridges should be designed to accommodate superelevation rates of 10 percent or less. See Index 202.2 for standard superelevation rates.

Superelevation transitions on bridges are difficult to construct and almost always result in an unsightly appearance of the bridge and the bridge railing.

Therefore, if possible, horizontal curves should begin and end a sufficient distance from the bridge so that no part of the superelevation transition extends onto the bridge.

Alignment and safety considerations, however, are paramount and must not be sacrificed to meet the above criteria.

Topic 204 - Grade

204.1 General Controls

The grade line is a reference line by which the elevation of the pavement and other features of the highway are established. It is controlled mainly by topography, type of highway, horizontal alignment, performance of heavy vehicles, right of way costs, safety, sight distance, construction costs, cultural development, drainage, and pleasing appearance.

All portions of the grade line must meet sight distance requirements for the design speed classification of the road.

In flat terrain, the elevation of the grade line is often controlled by drainage considerations. In rolling terrain, some undulation in the grade line is often advantageous for construction economy. This should be done with appearance in mind; for example, a grade line on tangent alignment exhibiting a series of humps visible for some distance ahead should be avoided whenever possible. In rolling hills or mountainous terrain, however, the grade line usually is more closely dependent upon physical controls.

In considering alternative profiles, economic comparisons involving earthwork quantities and/or retaining walls should be made. A balanced earthwork design is most cost effective. When long or steep grades are involved, economic comparisons should include vehicle operating costs.

The standards in Topic 204 also apply to portions of local streets and roads within the State right of way which connect directly to a freeway or expressway, or are expected to do so in the foreseeable future. **For local facilities which are within the State right of way and where there is no connection or the connection is to a non-controlled access facility (conventional highway), AASHTO standards shall prevail. If the local agency having**

jurisdiction over the local facility in question maintains standards that exceed AASHTO standards, then the local agency standards should prevail.

204.2 Position With Respect to Cross Section

The grade line should generally coincide with the axis of rotation for superelevation (see Index 202.4). Its relation to the cross section should be as follows:

- (1) *Undivided Highways.* The grade line should coincide with the highway centerline.
- (2) *Ramps and Freeway-to-freeway Connections.* Although the grade line is usually positioned at the left edge of traveled way, either edge of traveled way or centerline may be used on multilane facilities.
- (3) *Divided Highways.* The grade line should be positioned at the centerline of the median for paved medians 65 feet wide or less, thus avoiding a “saw tooth” section, which can reduce horizontal stopping sight distance.

The grade line may be positioned at the ultimate median edge of traveled way when:

- (a) The median edges of traveled way of the two roadways are at equal elevation.
- (b) The two roadways are at different elevations as described in Index 204.8.
- (c) The width of median is nonuniform (see Index 305.6).

204.3 Standards for Grade

Table 204.3 shows the maximum grades which shall not be exceeded for the condition indicated.

Steep grades affect truck speeds and overall capacity. They also cause operational problems at intersections. For these reasons it is desirable to provide the flattest grades practicable (see Index 204.5 for information on truck issues with grades).

Table 204.3

Maximum Grades for Type of Highway and Terrain Conditions

Type of Terrain	Freeways and Expressways	Rural Highways	Urban Highways
Level	3%	4%	6%
Rolling	4%	5%	7%
Mountainous	6%	7%	9%

Minimum grades should be 0.5 percent in snow country and 0.3 percent at other locations. Except for conventional highways in urban or suburban areas, a level grade line is permissible in level terrain where side fill slopes are 4:1 or flatter and dikes are not needed to carry water in the roadbed. Flat grades are not permissible in superelevation transitions due to flat spots which cause ponding on the roadbed.

Ramp grades should not exceed 8 percent. On descending on-ramps and ascending off-ramps, one percent steeper is allowed (see Index 504.2(5)).

204.4 Vertical Curves

Properly designed vertical curves should provide adequate sight distance, safety, comfortable driving, good drainage, and pleasing appearance.

A parabolic vertical curve is used. Figure 204.4 gives all necessary mathematical relations for computing a vertical curve, either at crests or sags. For algebraic grade differences of 2 percent and greater, and design speeds equal to or greater than 40 miles per hour, the minimum length of vertical curve in feet should be equal to 10V, where V = design speed. As an example, a 65 miles per hour design speed would require a 650-foot minimum vertical curve length. For algebraic grade differences of less than 2 percent, or design speeds less than 40 miles per hour, the vertical curve length should be a minimum of 200 feet. Vertical curves are not required where the algebraic difference in grades is 0.5 percent or less. Grade breaks should not be closer together than 50 feet and a total of all

- Deck-type structures: for simple spans use d/s (depth to span ratio) = 0.08; for continuous multiple span structures use d/s = 0.07. These ratios do not include the additional 2 feet required above the deck for ballast and rail height.
- (b) Highway Structures.
- Structures with single spans of 100 feet or less, use d/s = 0.06.
 - Structures with single spans between 100 feet and 180 feet use d/s = 0.045.
 - Continuous structures with multiple spans of 100 feet or less, use d/s = 0.055.
 - Continuous structures with multiple spans of more than 100 feet, use d/s = 0.04.
 - Geometric plans should be submitted to the DES – Structure Design prior to preparation of the Project Report so that preliminary studies can be prepared. Preliminary bridge type selection should be a joint effort between the DES – Structure Design and the District.
- (2) *Steel or Precast Concrete Structures.* Steel and precast concrete girders in lieu of cast-in-place concrete eliminate falsework, and may permit lower grade lines and reduced approach fill heights. Potential cost savings from elimination of falsework, lowered grade lines, and the ability to accommodate settlement beneath the abutments should be considered in structure type selection along with unit price, aesthetics, uniformity, and any other relevant factors. Note that grade lines at grade separations frequently need to be adjusted after final structure depths are determined (see Index 309.2(3)). Details of traffic handling and stage construction should be provided when the bridge site plan is submitted to the DES – Structure Design if the design or construction of the structure is affected (see Drafting and Plans Manual, Section 3-3.2).
- (3) *Depressed Grade Line Under Structures.* Bridge and drainage design will frequently be simplified if the low point in the grade line is set a sufficient distance from the intersection of the centerlines of the structure and the highway so that drainage structures clear the structure footings.
- (4) *Grade Line on Bridge Decks.* Vertical curves on bridge decks should provide a minimum fall of 0.05-foot per station. This fall should not extend over a length greater than 100 feet. The flattest allowable tangent grade should be 0.3 percent.
- (5) *Falsework.* In many cases, it is economically justified to have falsework over traffic during construction in order to have a support-free open area beneath the permanent structure. The elimination of permanent obstructions usually outweighs objections to the temporary inconvenience of falsework during construction.
- Because the width of traffic openings through falsework can, and oftentimes does, significantly affect costs, special care should be given to determining opening widths. The following should be considered: staging and traffic handling requirements, accommodation of pedestrians and bicyclists, the width of approach roadbed that will exist at the time the bridge is constructed, traffic volumes, needs of the local agencies, controls in the form of existing facilities, and the practical challenges of falsework construction.
- The normal width of traffic openings and required falsework spans are shown in Table 204.8.
- The normal spans shown in Table 204.8 are for anchored temporary K-rail. When temporary K-rail is not anchored, add 4 feet to normal span to include K-rail deflection.
- The minimum vertical falsework clearance over freeways and nonfreeways shall be 15 feet.** The following items should be considered:
- Mix, volume, and speed of traffic.

**Table 204.8
Falsework Span and Depth Requirements**

Facility to be Spanned	Minimum Normal Width of Traffic Opening (2)(3)(4)	Resulting Falsework Normal Span ⁽¹⁾	Depth of Superstructure ⁽⁵⁾			
			Up to 6 feet	Up to 8 feet	Up to 10 feet	Up to 12 feet
			Minimum Falsework Depth			
Freeway & Non Freeway	20'	28'	1'-9"	1'-10"	1'-10"	1'-10½"
	25'	33'	1'-10½"	2'-1'	2'-1'	2'-8½"
	32'	40'	2'-0"	2'-8½"	2'-9"	3'-0"
	37'	45'	2'-9"	2'-11½"	3'-0"	3'-3"
	40'	48'	3'-0"	3'-0"	3'-2½"	3'-3"
	49'	57'	3'-3"	3'-3½"	3'-3½"	3'-3½"
	52'	60'	3'-3"	3'-3½"	3'-3½"	3'-4"
	61'	69'	3'-5"	3'-5"	3'-7"	3'-7½"
	64'	72'	3'-5"	3'-7½"	3'-7½"	3'-8"
73'	81'	3'-6"	3'-9"	3'-9"	3'-9"	

NOTES:

- (1) Includes 8' for two temporary K-rails and 2' to center line of post including 3" clearance between K-rail and footing pad. This is for K-rail anchored to the pavement.
- (2) Approach roadway width measured normal to lanes. Use next highest width if the approach roadway width is not shown in the table.
- (3) Dependent upon the width of approach roadbed available at the time of bridge construction.
- (4) Clear vehicular opening between temporary railings.
- (5) See Index 204.8 for preliminary depth to span ratios. For more detailed information, contact the Division of Engineering Services, Structure Design and refer to the Bridge Design Aids.

large vehicles are expected, then the maximum width should be 45 feet.

- (c) When only one driveway serves a given property, in no case should the width of the driveway including the side slope distances exceed the property frontage.
 - (d) When more than one driveway is to serve a given property, the total width of all driveways should not exceed 70 percent of the frontage where such a frontage is 100 feet or less. Where the frontage is more than 100 feet, the total driveway width should not exceed 60 percent of the frontage. In either case, the width of the individual driveway should not exceed those given in the preceding paragraphs. Where more than one driveway is necessary to serve any one property, not less than 20 feet of full height curb should be provided between driveways. This distance between driveways also applies to projects where curbs and gutters are not to be placed.
 - (e) Certain urban commercial driveways may need to accommodate the maximum legal vehicle. The width will be determined by the use of truck turn templates.
- (5) *Surfacing.* Where curbs, gutters, and sidewalks are to be placed, driveways should be constructed of portland cement concrete. Where only curbs and gutters are to be placed and pedestrian traffic or adjacent improvements do not warrant concrete driveway construction, the driveway may be paved with the same materials used for existing surfacing on the property to be served.
- (6) *Pedestrian Access.* Where sidewalks traverse driveways, the sidewalk shall continue across the driveway to alert driveway users that they are crossing a pedestrian walkway, and must yield to pedestrians on the sidewalk. Driveway corner radii should also be minimized to encourage low-speed turns by motorized vehicles and bicycles. For accessibility requirements, see DIB 82. Provision of this feature, as indicated in the Standard Plans, may require the acquisition of

a construction easement or additional right of way. Assessment of these needs must be performed early enough in the design to allow time for acquiring any necessary permits or right of way. Additionally, designers should consider the following:

- In many cases providing the pathway along the back of the driveway will lower the elevation at the back of the sidewalk. Depending on grades behind the sidewalk the potential may exist for roadway generated runoff to enter private property. The need for features such as low berms within the construction easement, or installation of catch basins upstream of the driveway should be determined.

When there are no sidewalks or other pedestrian facilities that follow the highway, the designer may develop driveway details that eliminate the flatter portion along the back edge in lieu of using the Standard Plans for driveways. Refer to Topic 105 for additional information related to pedestrian facilities.

205.4 Driveways on Frontage Roads and in Rural Areas

On frontage roads and in rural areas where the maximum legal vehicle must be accommodated, standard truck-turn templates should be used to determine driveway widths where the curb or edge of traveled way is so close to the right of way line that a usable connection cannot be provided within the standard limits.

Where county or city regulations differ from the State's, it may be desirable to follow their regulations, particularly where jurisdiction of the frontage road will ultimately be in their hands.

Details for driveway construction are shown on the Standard Plans. For corner sight distance, see Index 405.1(2)(c).

Driveways connecting to State highways shall be paved a minimum of 20 feet from the edge of shoulder or to the edge of State right of way, whichever is less to minimize or eliminate gravel from being scattered on the highway and to provide a paved surface for vehicles and bicycles to

accelerate and merge. Where larger design vehicles are using the driveway (e.g., dump trucks, flat bed trucks, moving vans, etc.), extend paving so the drive wheels will be on a paved surface when accelerating onto the roadway. For paving at crossings with Class I bikeways (Bike Paths), see Index 1003.1(6)

205.5 Financial Responsibility

Reconstructing or relocating any access openings, private road connections, or driveways required by revisions to the State highway facility should be done at State expense by the State or its agents. Reconstruction or relocation requested by others should be paid for by the requesting party.

Topic 206 - Pavement Transitions

206.1 General Transition Standards

Pavement transition and detour standards should be consistent with the section having the features built to the highest design standards. The transition should be made on a tangent section whenever possible and should avoid locations with horizontal and vertical sight distance restrictions. Whenever feasible, the entire transition should be visible to the driver of a vehicle approaching the narrower section. The design should be such that intersections at grade within the transition area are avoided. For decision sight distance at lane drops, see Index 201.7.

206.2 Pavement Widening

- (1) *Through Lane Additions.* Where through lanes, climbing lanes, or passing lanes are added, the minimum recommended distance over which to transition traffic onto the additional width is 250 feet per lane. Figure 206.2 shows several examples of acceptable methods for adding a lane in each direction to a two-lane highway.
- (2) *Turning, Ramp, and Speed Change Lanes.* Transitions for lane additions, either for left or right turns or to add a lane to a ramp, should typically occur over a length of 120 feet. Lengths shorter than 120 feet are acceptable where design speeds are below 45 miles per hour or for conditions as stated in Index 405.2(2)(c).

Where insufficient median width is available to provide for left turn lanes, through traffic will have to be shifted to the outside. See Figures 405.2A, B and C for acceptable methods of widening pavement to provide for median turn lanes.

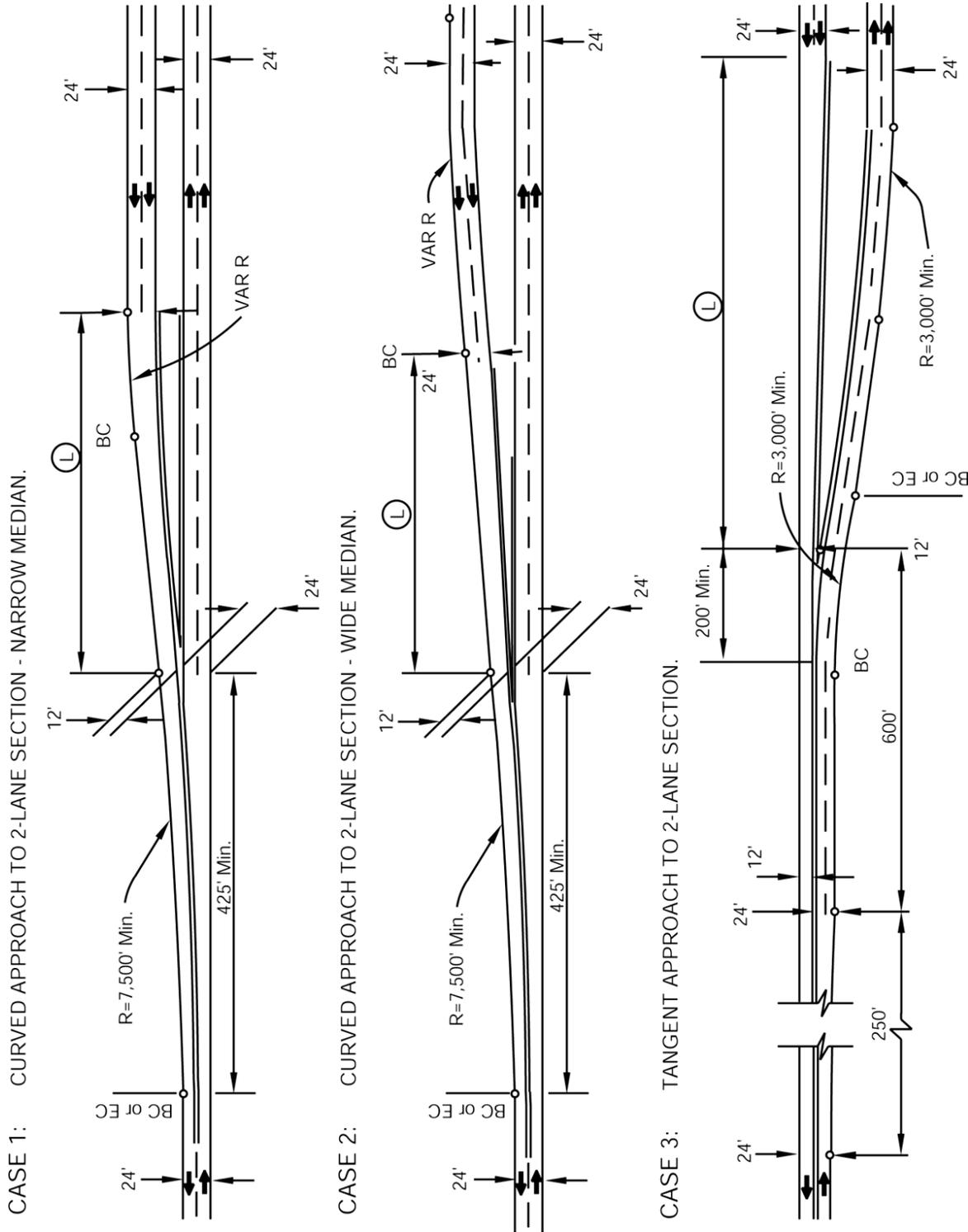
- (3) *Lane Widening.* An increase in lane width can occur at short radius curves which are widened for truck off-tracking, at ramp terminals with large truck turning volumes, or when new construction matches existing roadways with narrow lane widths. Extensive transition lengths are not necessary as the widening does not restrict the driver's expectations. Transition tapers for these types of situations should be at 10:1 (longitudinal to lateral).
- (4) *Shoulder and Bicycle Lane Widening.* Shoulder and bicycle lane widening should normally be accomplished in a manner that provides a smooth transition.

206.3 Pavement Reductions

- (1) *Through Lane Drops.* When a lane is to be dropped, it should be done by tapering over a distance equal to WV , where W = Width of lane to be dropped and V = Design Speed. In general, the transition should be on the right so that traffic merges to the left. Figure 206.2 provides several examples of acceptable lane drops at 4-lane to 2-lane transitions. The exception to using the WV criteria is for the lane drop/freeway merge movement on a branch connection which is accomplished using a 50:1 taper.
- (2) *Ramp and Speed Change Lanes.* As shown in Figures 504.2A and 504.3L, the standard taper for a ramp merge into a through traffic lane is 50:1 (longitudinal to lateral). Where ramp lanes are dropped prior to the merge with the through facility, the recommended taper is 50:1 for design speeds over 45 miles per hour, and the taper distance should be equal to WV for speeds below 45 miles per hour.

The "Ramp Meter Design Guidelines" also provide information on recommended and minimum tapers for ramp lane merges. These

Figure 206.2
Typical Two-lane to Four-lane Transitions



NOTE:
See Manual of Uniform Control Devices

EQUATION
 $L = WV$
 Where L = Length of variable width traveled way - feet.
 V = Design speed in mph
 W = Lane Width - feet.

guideline values are typically used in retrofit or restricted right-of-way situations, and are acceptable for the specific conditions stated in the guidelines.

Figure 405.9 shows the standard taper to be used for dropping an acceleration lane at a signalized intersection. This taper can also be used when transitioning median acceleration lanes.

Figures 405.2A, B and C show the recommended methods of transitioning pavement back into the median area on conventional highways after the elimination of left-turn lanes.

- (3) *Lane Reductions.* At any location where lane widths are being reduced, the minimum length over which to accomplish the transition should be equal to WV. See Index 504.6 for mainline lane reductions at interchanges.
- (4) *Shoulder Reduction.* Shoulder reductions should typically occur over a length equal to $\frac{3}{4}WV$. However, when shoulder widths are being reduced in conjunction with a lane addition or widening (as in Alt. A of Figure 504.3K), the shoulder reduction should be accomplished over the same distance as the addition or widening.

206.4 Temporary Freeway Transitions

It is highly desirable that the design standards for a temporary transition between the end of a freeway construction unit and an existing highway should not change abruptly from the freeway standards. Temporary freeway transitions must be reviewed by the Design Coordinator.

Topic 207 - Airway-Highway Clearances

207.1 Introduction

- (1) *Objects Affecting Navigable Airspace.* An object is considered an obstruction to air navigation if any portion of that object is of a height greater than the approach and transverse surfaces extending outward and upward from the airport runway. These objects include overhead signs, light

standards, moving vehicles on the highway and overcrossing structures, equipment used during construction, and plants.

- (2) *Reference.* The Federal Aviation Administration (FAA) has published a Federal Aviation Regulation (FAR) relative to airspace clearance entitled, "FAR Part 77, Obstructions Affecting Navigable Airspace", dated March 1993. This is an approved reference to be used in conjunction with this manual.

207.2 Clearances

- (a) Civil Airports--See Figure 207.2A.
- (b) Heliports--See Figure 207.2B.
- (c) Military Airports--See Figure 207.2C.
- (d) Navy Carrier Landing Practice Fields--See Figure 207.2D.

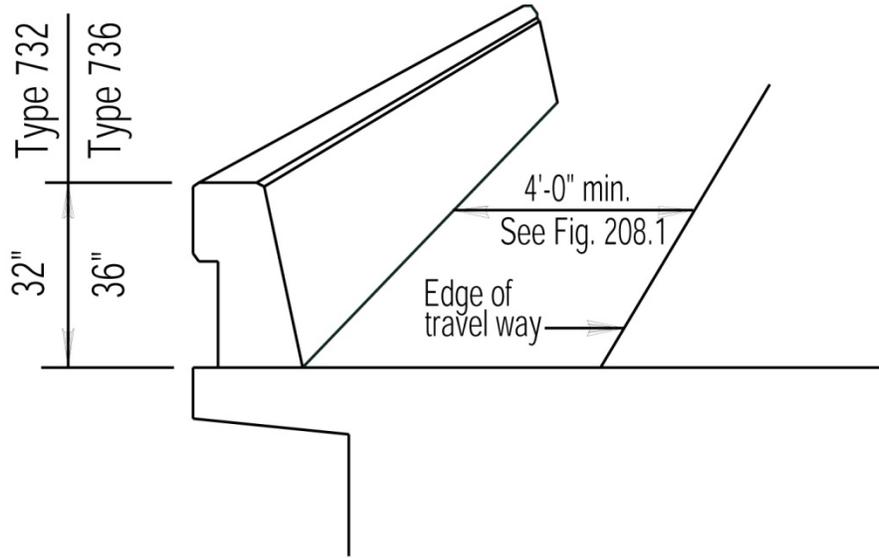
207.3 Submittal of Airway-Highway Clearance Data

The following procedure must be observed in connection with airway-highway clearances in the vicinity of airports and heliports.

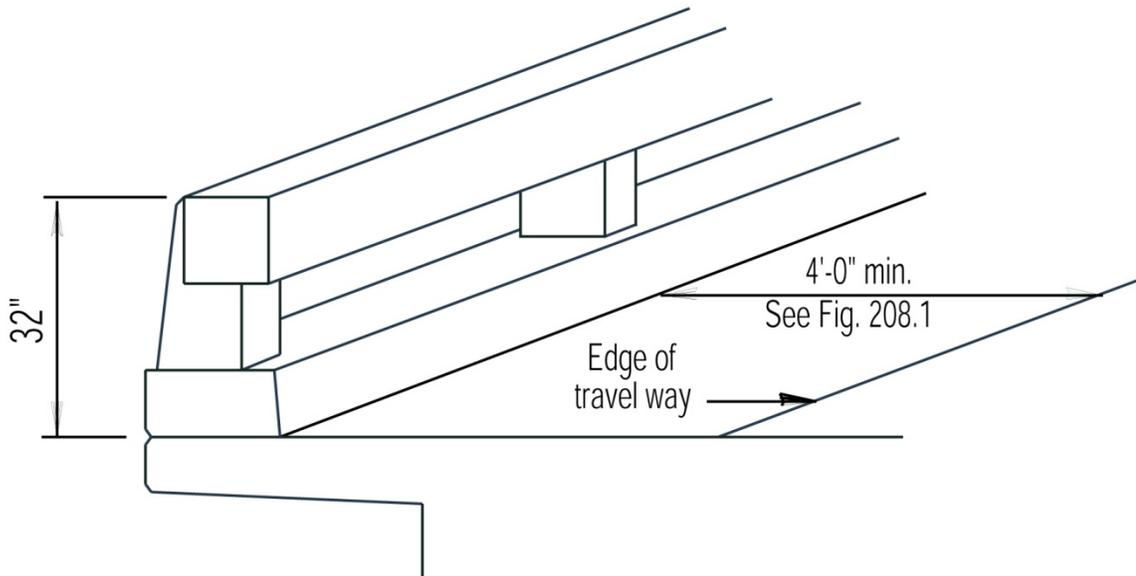
Notice to the FAA is required when highway construction is planned near an airport (civil or military) or a heliport. A "Notice of Proposed Construction or Alteration" should be submitted to the FAA Administrator when required under criteria listed in Paragraph 77.13 of the latest Federal Aviation Regulations, Part 77. Such notice should be given as soon as highway alignment and grade are firmly established. It should be noted that these requirements apply to both permanent objects and construction equipment. When required, four copies of FAA Form 7460-1, "Notice of Proposed Construction", and accompanying scaled maps must be sent to the FAA, Western-Pacific Regional Office, Chief-Air Traffic Division, AWP-520, 15000 Aviation Boulevard, Hawthorne, CA90260. Copies of FAA Form 7460-1 may be obtained from the FAA, Western-Pacific Regional Office or Caltrans, Division of Aeronautics.

The scaled maps accompanying FAA Form 7460-1 should contain the following minimum information.

Figure 208.10A
Vehicular Railings for
Bridge Structures

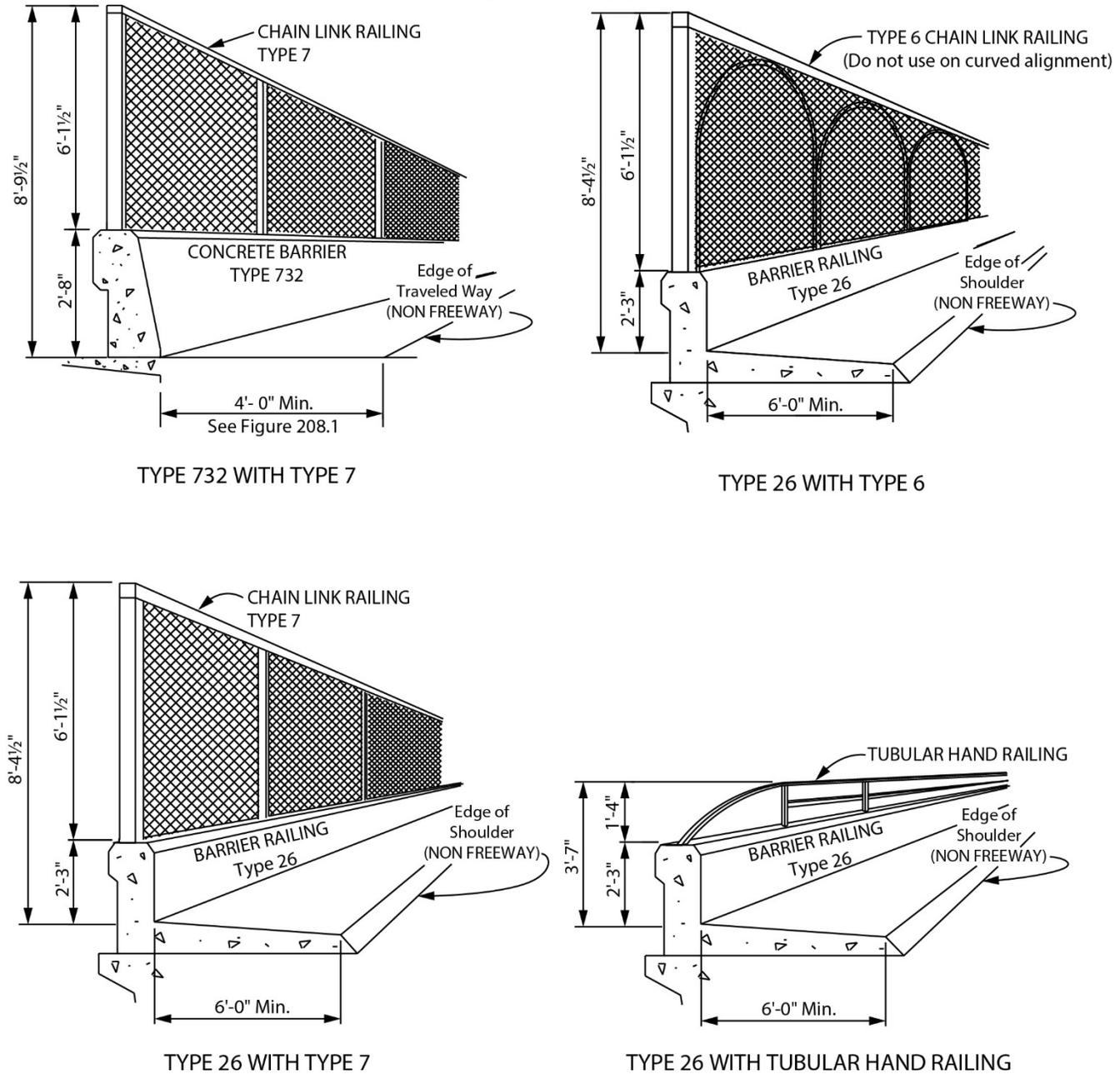


CONCRETE BARRIERS TYPE 732 AND TYPE 736



CONCRETE BARRIER TYPE 80

Figure 208.10B
Combination Vehicular Barrier and Pedestrian Railings for Bridge Structures



CHAPTER 300 GEOMETRIC CROSS SECTION

The selection of a cross section is based upon the joint use of the transportation corridor by vehicles, including trucks, public transit, cyclists and pedestrians. Designers should recognize the implications of this sharing of the transportation corridor and are encouraged to consider not only vehicular movement, but also movement of people, distribution of goods, and provision of essential services. Designers need also to consider the plan for the future of the route, consult Transportation Concept Reports for state routes.

Topic 301 - Traveled Way Standards

The traveled way width is determined by the number of lanes required to accommodate operational needs, terrain, safety and other concerns. The traveled way width includes the width of all lanes and bike lanes, but does not include the width of shoulders, sidewalks, curbs, dikes, gutters, or gutter pans. See Topic 307 for State highway cross sections, and Topic 308 for road cross sections under other jurisdictions.

Index 301.1 – Lane Width

The minimum lane width on two-lane and multilane highways, ramps, collector-distributor roads, and other appurtenant roadways shall be 12 feet, except as follows:

- **For conventional State highways with posted speeds less than or equal to 40 miles per hour and AADTT (truck volume) less than 250 per lane that are in urban, city or town centers (rural main streets), the minimum lane width shall be 11 feet.** The preferred lane width is 12 feet. See Index 81.3 for place type definitions.

Where a 2-lane conventional State highway connects to a freeway within an interchange, the lane width shall be 12 feet.

Where a multilane State highway connects to a freeway within an interchange, the

outer most lane of the highway in each direction of travel shall be 12 feet.

- For highways, ramps, and roads with curve radii of 300 feet or less, widening due to offtracking in order to minimize bicycle and vehicle conflicts must be considered. See Index 404.1 and Table 504.3A.
- For lane widths on roads under other jurisdictions, see Topic 308.

301.2 Class II Bikeway (Bike Lane) Lane Width

(1) *General.* **Class II bikeways (bike lanes), for the preferential use of bicycles, may be established within the roadbed and shall be located immediately adjacent to a traffic lane as allowed in this manual.** Typical Class II bikeway configurations are illustrated in Figure 301.2A. A bikeway located behind on-street parking, physical separation, or barrier within the roadway is not a Class II bikeway (bike lane); see Index 1003.1 Class I Bikeway (Bike Path) for standards and design guidance. **The minimum Class II bike lane width shall be 4 feet, except where:**

- Adjacent to on-street parking, the minimum bike lane should be 5 feet.
- Posted speeds are greater than 40 miles per hour, the minimum bike lane should be 6 feet, or
- **On highways with concrete curb and gutter, a minimum width of 3 feet measured from the bike lane stripe to the joint between the shoulder pavement and the gutter shall be provided.**

Class II bikeways may be included as part of the shoulder width See Topic 302.

As grades increase, downhill bicycle speeds can increase, which increases the width needed for the comfort of bicycle operation. If bicycle lanes are to be marked, additional bike lane width is recommended to accommodate these higher bicycle speeds. See Index 204.5(4) for guidance on accommodating bicyclists on uphill grades where a Class II bikeway is not included.

If bike lanes are to be located on one-way streets, they may be placed on either or both sides of the street. When only one bicycle lane is provided, it should be located on the side of the street that presents the lowest number of conflicts for bicyclists which facilitates turning movements and access to destinations on the street.

- (2) *On-Street Parking Adjacent to Class II Bikeways.* Parking adjacent to bike lanes is discussed in subsection (1) above and addressed in Table 302.1, Note (7). Part-time bike lanes with part-time on-street parking is discouraged. This type of bike lane may only be considered if the majority of bicycle travel occurs during the hours of parking prohibition. When such an installation is being considered refer to the California MUTCD and traffic operations for direction regarding proper signing and marking.
- (3) *Reduction of Cross Section Elements Adjacent to Class II Bikeways.* There are situations where it may be desirable to reduce the width of the lanes in order to add or widen bike lanes or shoulders. In determining the appropriateness of narrower traffic lanes, consideration should be given to factors such as motor vehicle speeds, truck volumes, alignment, bike lane width, sight distance, and the presence of on-street parking. When on-street parking is permitted adjacent to a bike lane, or on a shoulder where bicycling is not prohibited, reducing the width of the adjacent traffic lane may allow for wider bike lanes or shoulders, to provide greater clearance between bicyclists and driver-side doors when opened.

301.3 Cross Slopes

- (1) *General.* The purpose of sloping on roadway cross sections is to provide a mechanism to direct water (usually from precipitation) off the traveled way. Undesirable accumulations of water can lead to hydroplaning or other problems which can increase accident potential. See Topics 831 and 833 for hydroplaning considerations. For roadways with three (3) lanes or more sloped in the same direction, see topic 833.2.

(2) *Standards.*

- (a) **The standard cross slope to be used for new construction on the traveled way for all types of surfaces shall be 2 percent.**
- (b) **For resurfacing or widening (only when necessary to match existing cross slope), the minimum shall be 1.5 percent and the maximum shall be 3 percent.** However, the cross slope on 2-lane and multilane HMA highways should be increased to 2 percent if the cost is reasonable.
- (c) **On unpaved roadway surfaces, including gravel and penetration treated earth, the cross slope shall be 2.5 percent to 5.0 percent.**

On undivided highways with two or more lanes in a normal tangent section, the high point of the crown should be centered on the pavement and the pavement sloped toward the edges on a uniform grade.

For rehabilitation and widening projects, the maximum algebraic difference in cross slope between adjacent lanes of opposing traffic for either 2-lane or undivided multilane highways should be 6 percent. For new construction, the maximum shall be 4 percent.

On divided highway roadbeds, the high point of crown may be centered at, or left of, the center of the traveled way, and preferably over a lane line (tangent sections). This strategy may be employed when adding lanes on the inside of divided highways, or when widening an existing "crowned" 2-lane highway to a 4-lane divided highway by utilizing the existing 2-lane pavement as one of the divided highway roadbeds.

The maximum algebraic difference in cross slope between same direction traffic lanes of divided highway roadbeds should be 4 percent.

The maximum difference in cross slope between the traveled way and the shoulder should not exceed 8 percent. This applies to

and moisture. Topsoil should be reapplied to stepped slopes to encourage revegetation.

For appearance, steps on small cuts viewed from the roadway should be cut parallel to the road grade. Runoff is minimized on steps cut parallel to roads with grades up to 10 percent, as long as the natural ravel from construction is left on the steps. Steps less than one-half full should not be cleaned.

High cuts viewed from surrounding areas should be analyzed before a decision is made to form steps parallel to the roadway or horizontal. In some cases, horizontal steps may be more desirable. Special study is also necessary when a sag occurs in the vertical alignment within the cut. In all cases at the ends of cuts, the steps should wrap around the rounded transition.

The detail or contract special provisions should allow about a 20 percent variation, expressed in terms of tenths of a foot. Some irregularity will improve the appearance of the slope by making it appear more natural.

In designing step width, the material's weathering characteristics should generally be considered. Widths over approximately 2 feet should be avoided because of prominence and excessive time to achieve a weathered and natural appearance. Contact the DES-GS and the District Landscape Architect for more information about the width of steps.

Topic 305 - Median Standards

305.1 Width

Median width is expressed as the dimension between inside edges of traveled way, including the inside shoulder. This width is dependent upon the type of facility, costs, topography, and right of way. Consideration may be given to the possible need to construct a wider median than prescribed in Cases (1), (2), and (3), below, in order to provide for future expansion to accommodate:

- (a) Public Transit (rail and bus).
- (b) Traffic needs more than 20 years after completion of construction.

Median width as presented in Case (1) below applies to new construction, projects to increase mainline capacity and to reconstruction projects.

Any recommendation to provide additional median width should be identified and documented as early as possible and must be justified in a Project Study Report and/or Project Report. Attention should be given to such items as initial costs, future costs for outside widening, the likelihood of future needs for added mixed flow or High-Occupancy Vehicle (HOV) lanes, traffic interruption, future mass transit needs and right of way considerations. (For instance, increasing median width may add little to the cost of a project where an entire city block must be acquired in any event.)

Median pedestrian refuge areas at intersections lessen the risk of pedestrian exposure to traffic. See Index 405.4(3) and DIB 82 for pedestrian refuge guidance.

If additional width is justified, the minimum median widths provided below should be increased accordingly.

Minimum median widths for the design year (as described below) should be used in order to accommodate the ultimate highway facility (type and number of lanes):

(1) *Freeways and Expressways.*

- (a) Urban Areas. Where managed lanes (HOV, Express, etc) or transit facilities are planned, the minimum median width should be 62 feet. Where there is little or no likelihood of managed lanes or transit facilities planned for the future, the minimum median width should be 46 feet. However, where physical and economic limitations are such that a 46-foot median cannot be provided at reasonable cost, the minimum median width for freeways and expressways in urban areas should be 36 feet.

- (b) Rural Areas. The minimum median width for freeways and expressways in rural areas should be 62 feet.

(2) *Conventional Highways.* Appropriate median widths for non-controlled access highways vary widely with the type of facility being designed. In Urban and Rural Main Street

areas, the minimum median width for multilane conventional highways should be 12 feet. However, this width would not provide room for left-turn lanes at intersections with raised curb medians, nor left-turn lanes in striped medians with room for pedestrian refuge areas. Posted speed and left shoulder width can also affect median width. See Table 302.1.

Medians refuge areas at pedestrian crosswalks and bicycle path crossings provide a space for pedestrians and bicyclists. They allow these users to cross one direction of traffic at a time. Where medians are provided, they should allow access through them for pedestrians and bicyclists as necessary. Bicycle crossings through paved medians should line up with the bicycle path of travel and not require bicyclists to utilize the pedestrian crosswalk. See Index 405.4 for additional requirements.

Where medians are provided for proposed future two-way left-turn lanes, median widths up to 14 feet may be provided to conform to local agency standards (see Index 405.2). **In rural areas the minimum median width for multilane conventional highways shall be 12 feet.** This provides the minimum space necessary to accommodate a median barrier and 5-foot shoulders. Whenever possible, and where it is appropriate, this minimum width should be increased to 30 feet or greater.

At locations where a climbing or passing lane is added to a 2-lane conventional highway, a 4-foot median (or “soft barrier”) between opposing traffic lanes should be used.

(3) *Facilities under Restrictive Conditions.* Where certain restrictive conditions, including steep mountainous terrain, extreme right of way costs, and/or significant environmental factors are encountered, the basic median widths above may not be attainable. Where such conditions exist, a narrower median, down to the limits given below, may be allowed with adequate justification. (See Index 307.5.)

(a) Freeways and Expressways. **In areas where restrictive conditions prevail the minimum median width shall be 22 feet.**

(b) Conventional Highways. Median widths should be consistent with requirements for two-way left-turn lanes or the need to construct median barriers (as discussed in Index 305.1(2)), but may be reduced or eliminated entirely in extreme situations.

The above stated minimum median widths should be increased at spot locations to accommodate the construction of bridge piers or other planned highway features while maintaining standard cross section elements such as inside shoulder width and horizontal clearance. If a bridge pier is to be located in a tangent section, the additional width should be developed between adjacent horizontal curves; if it is to be located in a curve, then the additional width should be developed within the limits of the curve. Provisions should be made for piers 6 feet wide or wider. Median widths in areas of multilevel interchanges or other major structures should be coordinated with the Division of Engineering Services, Structures Design (DES-SD).

Consideration should also be given to increasing the median width at unsignalized intersections on expressways and divided highways in order to provide a refuge area for large trucks attempting to cross the State route.

In any case, the median width should be the maximum attainable at reasonable cost based on site specific considerations of each project.

See Index 613.5(2)(b) for paved median pavement structure requirements.

305.2 Median Cross Slopes

Unsurfaced medians up to 65 feet wide should be sloped downward from the adjoining shoulders to form a shallow valley in the center. Cross slopes should be 10:1 or flatter; 20:1 being preferred. Slopes as steep as 6:1 are acceptable in exceptional cases when necessary for drainage, stage construction, etc. Cross slopes in medians greater than 65 feet should be treated as separate roadways (see Index 305.6).

Paved medians, including those bordered by curbs, should be crowned at the center, sloping towards the sides at the slope of the adjacent pavement.

305.3 Median Barriers

See Chapter 7 of the Traffic Manual.

305.4 Median Curbs

See Topic 303 for curb types and usage in medians and Index 405.5(1) for curbs in median openings.

305.5 Paved Medians

(1) *Freeways.*

- (a) 6 or More Lanes--Medians 30 feet wide or less should be paved.
- (b) 4 Lanes--Medians 22 feet or less in width should be paved. Medians between 22 feet and 30 feet wide should be paved only if a barrier is installed. With a barrier, medians wider than 30 feet should not normally be paved.

Where medians are paved, each half generally should be paved in the same plane as the adjacent traveled way.

- #### (2) *Nonfreeways.* Unplanted curbed medians generally are to be surfaced with minimum 0.15 foot of Portland cement concrete.

For additional information on median cross slopes see Index 305.2.

305.6 Separate Roadways

- (1) *General Policy.* Separate grade lines are not considered appropriate for medians less than 65 feet wide (see Index 204.7).
- (2) *Median Design.* The cross sections shown in Figure 305.6 with a 23-foot graded area left of traffic are examples of median treatment to provide maneuvering room for out-of-control users. This optional treatment may be used where extra recovery area is desired (see Index 307.6).

See Index 302.1 for shoulder widths and Index 302.2 for shoulder cross slopes.

Topic 306 - Right of Way

306.1 General Standards

The right of way widths for State highways, including frontage roads to be relinquished, should provide for installation, operation and maintenance

of all cross section elements needed depending upon the type of facility, including median, traffic lanes, bicycle lanes, outside shoulders, sidewalks, recovery areas, slopes, sight lines, outer separations, ramps, walls, transit facilities and other essential highway appurtenances. For minimum clearance from the right of way line to the catch point of a cut or fill slope, see Index 304.2. Fixed minimum widths of right of way, except for 2-lane highways, are not specified because dimensions of cross-sectional elements may require narrow widths, and right of way need not be of constant width. The minimum right of way width on new construction for 2-lane highways should be 150 feet.

306.2 Right of Way Through the Public Domain

Right of way widths to be obtained or reserved for highway purposes through lands of the United States Government or the State of California are determined by laws and regulations of the agencies concerned.

Topic 307 - Cross Sections for State Highways

307.1 Cross Section Selection

The cross section of a State highway is based upon the number of vehicles, including trucks, buses, bicycles, and safety, terrain, transit needs and pedestrians. Other factors such as sidewalks, bike paths and transit facilities, both existing and future should be considered. For 2-lane roads the roadbed width is influenced by the factors discussed under Index 307.2. The roadbed width for multilane facilities should be adequate to provide capacity for the design hourly volume based upon capacity considerations discussed under Index 102.1.

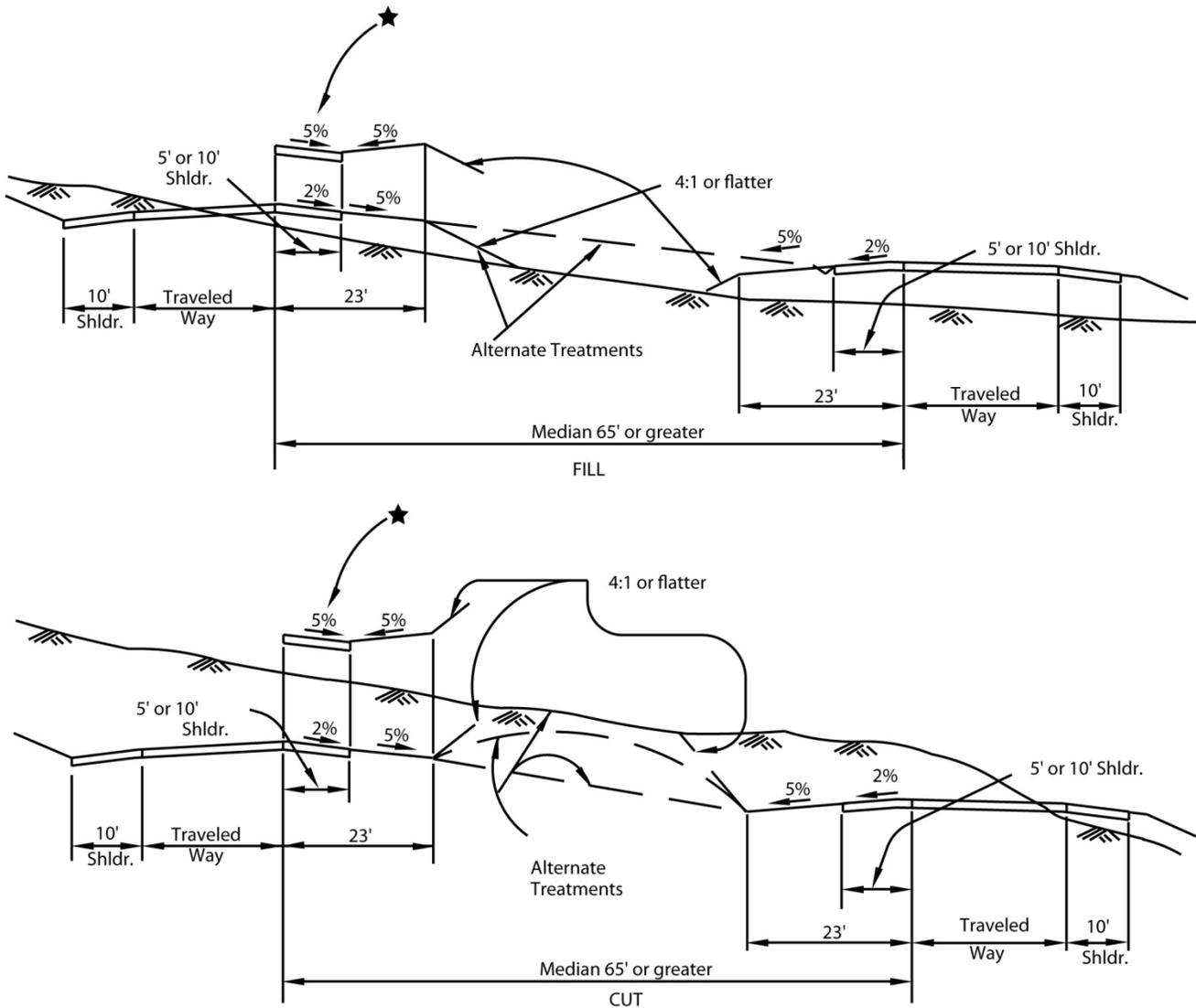
307.2 Two-lane Cross Sections for New Construction

These standards are to be used for highways on new alignment as well as on existing highways where the width, alignment, grade, or other geometric features are being upgraded.

A 2-lane, 2-way roadbed consists of a 24-foot wide traveled way plus paved shoulders. In order to provide structural support, the minimum paved

Figure 305.6

Optional Median Designs for Freeways with Separate Roadways



NOTES:

Left Paved Shoulder Width
 10' for 6-lane and 8-lane roadways
 5' for 4-lane roadways

Side Slopes
 See Index 304.1

★ Superelevated section

(1) Geometric Configurations

- (a) Crossing-Type Intersections - “Tee” and 4-legged intersections
- (b) Circular Intersections –roundabouts, traffic circles, rotaries; however, only roundabouts are acceptable for State highways.
- (c) Alternative Intersection Designs – various effective geometric alternatives to traditional designs that can reduce crashes and their severity, improve operations, reduce congestion and delay typically by reducing or altering the number of conflict points; these alternatives include geometric design features such as intersections with displaced left-turns or variations on U-turns.

(2) Intersection Control strategies, See California MUTCD and Traffic Operations Policy Directive (TOPD) Number 13-02, Intersection Control Evaluation for procedures and guidance on how to evaluate, compare and select from among the following intersection control strategies:

- (a) Two-Way Stop Controlled - for minor road traffic
- (b) All-Way Stop Control
- (c) Signal Control
- (d) Yield Control (Roundabout)

Historically, crossing-type intersections with signal or “STOP”-control have been used on the State highway system. However, other intersection types, given the appropriate circumstances may enhance intersection performance through fewer or less severe crashes and improve operations by reducing overall delay. Alternative intersection geometric designs should be considered and evaluated early in the project scoping, planning and decision-making stages, as they may be more efficient, economical and safer solutions than traditional designs. Alternative intersection designs can effectively balance the safety and mobility needs of the motor vehicle drivers, transit riders, bicyclists and pedestrians using the intersection.

401.6 Transit

Transit use may range from periodic buses, handled as part of the normal mix of vehicular traffic, to Bus

Rapid Transit (BRT) or light rail facilities which can have a large impact on other users of the intersection. Consideration of these modes should be part of the early planning and design of intersections.

Topic 402 - Operational Features Affecting Design**402.1 Capacity**

Adequate capacity to handle peak period traffic demands is a basic goal of intersection design.

(1) Unsignalized Intersections. The “Highway Capacity Manual”, provides methodology for capacity analysis of unsignalized intersections controlled by “STOP” or “YIELD” signs. The assumption is made that major street traffic is not affected by the minor street movement. Unsignalized intersections generally become candidates for signalization when traffic backups begin to develop on the cross street or when gaps in traffic are insufficient for drivers to yield to crossing pedestrians. See the California MUTCD, for signal warrants. Changes to intersection controls must be coordinated with District Traffic Branch.

(2) Signalized Intersections. See Topic 406 for analysis of simple signalized intersections, including ramps. The analysis of complex and alternative intersections should be referred to the District Traffic Branch; also see Traffic Operations Policy Directive (TOPD) Number 13-02.

(3) Roundabout Intersections. See TOPD Number 13-02 for screening process and the Intersection Control Evaluation(ICE) Process Informational Guide for operational analysis methods and tools.

402.2 Collisions

(1) General. Intersections have a higher potential for conflict compared to other sections of the highway because travel is interrupted, traffic streams cross, and many types of turning movements occur.

The type of traffic control affects the type of collisions. Signalized intersections tend to have more rear end and same-direction

sideswipes than intersections with “STOP”-control on minor legs. Roundabouts experience few angle or crossing collisions. Roundabouts reduce the frequency and severity of collisions, especially when compared to the performance of signalized intersections in high speed environments. Other alternative intersection types are configurations to consider for minimizing the number of conflict points.

(2) *Undesirable Geometric Features.*

- Inadequate approach sight distance.
- Inadequate corner sight distance.
- Steep grades.
- Five or more approaches.
- Presence of curves within intersections(unless at roundabouts).
- Inappropriately large curb radii.
- Long pedestrian crossing distances.
- Intersection Angle <75 degrees (see Topic 403).

402.3 On-Street Parking

On-street parking generally decreases through-traffic capacity, impedes traffic flow, and increases crash potential. Where the primary service of the arterial is the movement of vehicles, it may be desirable to prohibit on-street parking on State highways in urban and suburban expressways and rural arterial sections. However, within urban and suburban areas and in rural communities located on State highways, on-street parking should be considered in order to accommodate existing land uses. Where adequate off-street parking facilities are not available, the designer should consider on-street parking, so that the proposed highway improvement will be compatible with the land use. On-street parking as well as off-street parking needs to comply with DIB82. See AASHTO, A Policy on Geometric Design of Highways and Streets for additional guidance related to on-street parking.

402.4 Consider All Users

Intersections should accommodate all users of the facility, including vehicles, bicyclists, pedestrians and transit. Bicycles have all the rights and responsibilities as motorist per the California

Vehicle Code, but should have separate consideration of their needs, even separate facilities if volumes warrant. Pedestrians should not be prohibited from crossing one or more legs of an intersection, unless no other safe alternative exists. Pedestrians can be prohibited from crossing one or more legs of an intersection if a reasonable alternate route exists and there is a demonstrated need to do so. All pedestrian facilities shall be ADA compliant as outlined in DIB 82. Transit needs should be determined early in the planning and design phase as their needs can have a large impact on the performance of an intersection. Transit stops in the vicinity of intersections should be evaluated for their effect on the safety and operation of the intersection(s) under study. See Topic 108 for additional information.

402.5 Speed-Change Areas

Speed-change areas for vehicles entering or leaving main streams of traffic are beneficial to the safety and efficiency of an intersection. Entering traffic merges most efficiently with through traffic when the merging angle is less than 15 degrees and when speed differentials are at a minimum.

Topic 403 - Principles of Channelization

403.1 Preference to Major Movements

The provision of direct free-flowing high-standard alignment to give preference to major movements is good channelization practice. This may require some degree of control of the minor movements such as stopping, funneling, or even eliminating them. These controlling measures should conform to natural paths of movement and should be introduced gradually to promote smooth and efficient operation.

403.2 Areas of Conflict

Large multilane undivided intersection areas are undesirable. The hazards of conflicting movements are magnified when motorists, bicyclists, and pedestrians are unable to anticipate movements of other users within these areas. Channelization reduces areas of conflict by separating or regulating traffic movements into definite paths of travel by the use of pavement markings or traffic islands.

Multilane undivided intersections, even with signalization, are more difficult for pedestrians to cross. Providing pedestrian refuge islands enable pedestrians to cross fewer lanes at a time.

See Index 403.7 for traffic island guidance when used as pedestrian refuge. Curb extensions shorten crossing distance and increase visibility. See Index 303.4 for curb extensions.

403.3 Angle of Intersection

A right angle (90°) intersection provides the most favorable conditions for intersecting and turning traffic movements. Specifically, a right angle provides:

- The shortest crossing distance for motor vehicles, bicycles, and pedestrians.
- Sight lines which optimize corner sight distance and the ability of motorists to judge the relative position and speed of approach traffic.
- Intersection geometry that can reduce vehicle turning speeds so collisions are more easily avoided and the severity of collisions are minimized.
- Intersection geometry that sends a message to turning bicyclists and motorists that they are making a turning movement and should yield as appropriate to through traffic on the roadway they are leaving, to traffic on the receiving roadway, and to pedestrians crossing the intersection.

Minor deviations from right angles are generally acceptable provided that the potentially detrimental impact on visibility and turning movements for large trucks (see Topic 404) can be mitigated. However, large deviations from right angles may decrease visibility, hamper certain turning operations, and will increase the size of the intersection and therefore crossing distances for bicyclists and pedestrians, may encourage high speed turns, and may reduce yielding by turning traffic. When a right angle cannot be provided due to physical constraints, the interior angle should be designed as close to 90 degrees as is practical, but should not be less than 75 degrees. Mitigation should be considered for the affected intersection design features. (See Figure 403.3A). A 75 degree angle does not unreasonably increase the crossing distance or generally decrease visibility. Class II

bikeway crossings at railroads follow similar guidance to Class I bikeway crossings at railroads, see Index 1003.5(3), and Figure 403.3B.

A characteristic of skewed intersection angles is that they result in larger intersections.

When existing intersection angles are less than 75 degrees, the following retrofit improvement strategies should be considered:

- Realign the subordinate intersection legs if the new alignment and intersection location(s) can be designed without introducing new geometric or operational deficiencies.
- Provide acceleration lanes for difficult turning movements due to radius or limited visibility.
- Restrict problematic turning movements; e.g. for minor road left turns with potentially limited visibility.
- Provide refuge areas for pedestrians at very long crossings.

For additional guidance on the above and other improvement strategies, consult with the HQ Design Reviewer or HQ Traffic Liaison.

Particular attention should be given to skewed angles on curved alignment with regards to sight distance and visibility. Crossroads skewed to the left have more restricted visibility for drivers of vans and trucks than crossroads skewed to the right. In addition, severely skewed intersection angles, coupled with steep downgrades (generally over 4 percent) can increase the potential for high centered vehicles to overturn where the vehicle is on a downgrade and must make a turn greater than 90 degrees onto a crossroad. These factors should be considered in the design of skewed intersections.

403.4 Points of Conflict

Channelization separates and clearly defines points of conflict within the intersection. Bicyclists, pedestrians and motorists should be exposed to only one conflict or confronted with one decision at a time.

Speed-change areas for diverging traffic should provide adequate length clear of the through lanes to permit vehicles to decelerate after leaving the through lanes.

March 7, 2014

See AASHTO, A Policy on Geometric Design of Highways and Streets for additional guidance on speed-change lanes.

Figure 403.3A
Angle of Intersection
(Minor Leg Skewed to the Right)

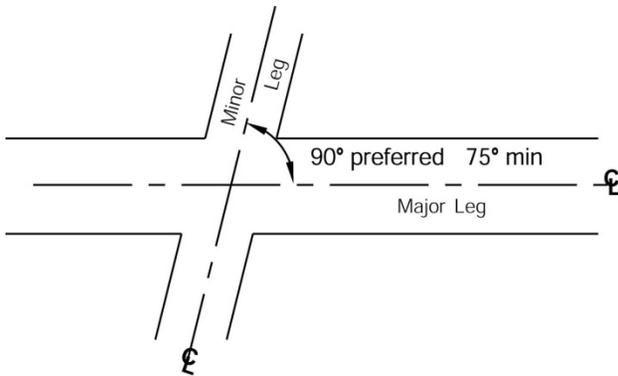
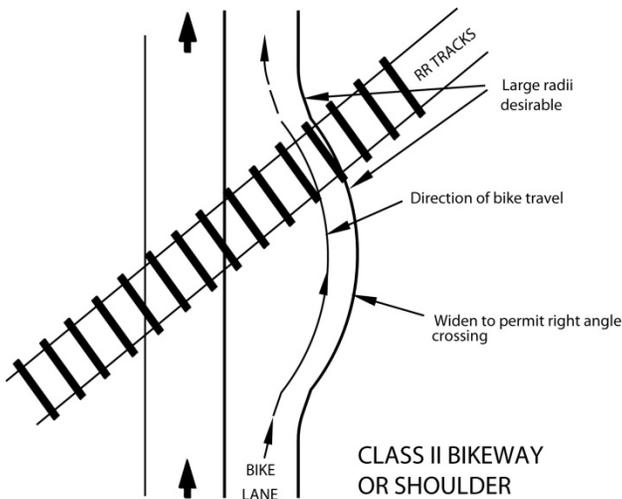


Figure 403.3B
Class II Bikeway
Crossing Railroad



403.5 (Currently Not In Use)

403.6 Turning Traffic

A separate turning lane removes turning movements from the intersection area. Abrupt changes in alignment or sight distance should be avoided, particularly where traffic turns into a separate turning lane from a high-standard through facility.

For wide medians, consider the use of offset left-turn lanes at both signalized and unsignalized intersections. Opposing left-turn lanes are offset or shifted as far to the left as practical by reducing the width of separation immediately before the intersection. Rather than aligning the left-turn lane exactly parallel with and adjacent to the through lane, the offset left-turn lane is separated from the adjacent through lane. Offset left-turn lanes provide improved visibility of opposing through traffic. For further guidance on offset left-turn lanes, see AASHTO, A Policy on Geometric Design of Highways and Streets.

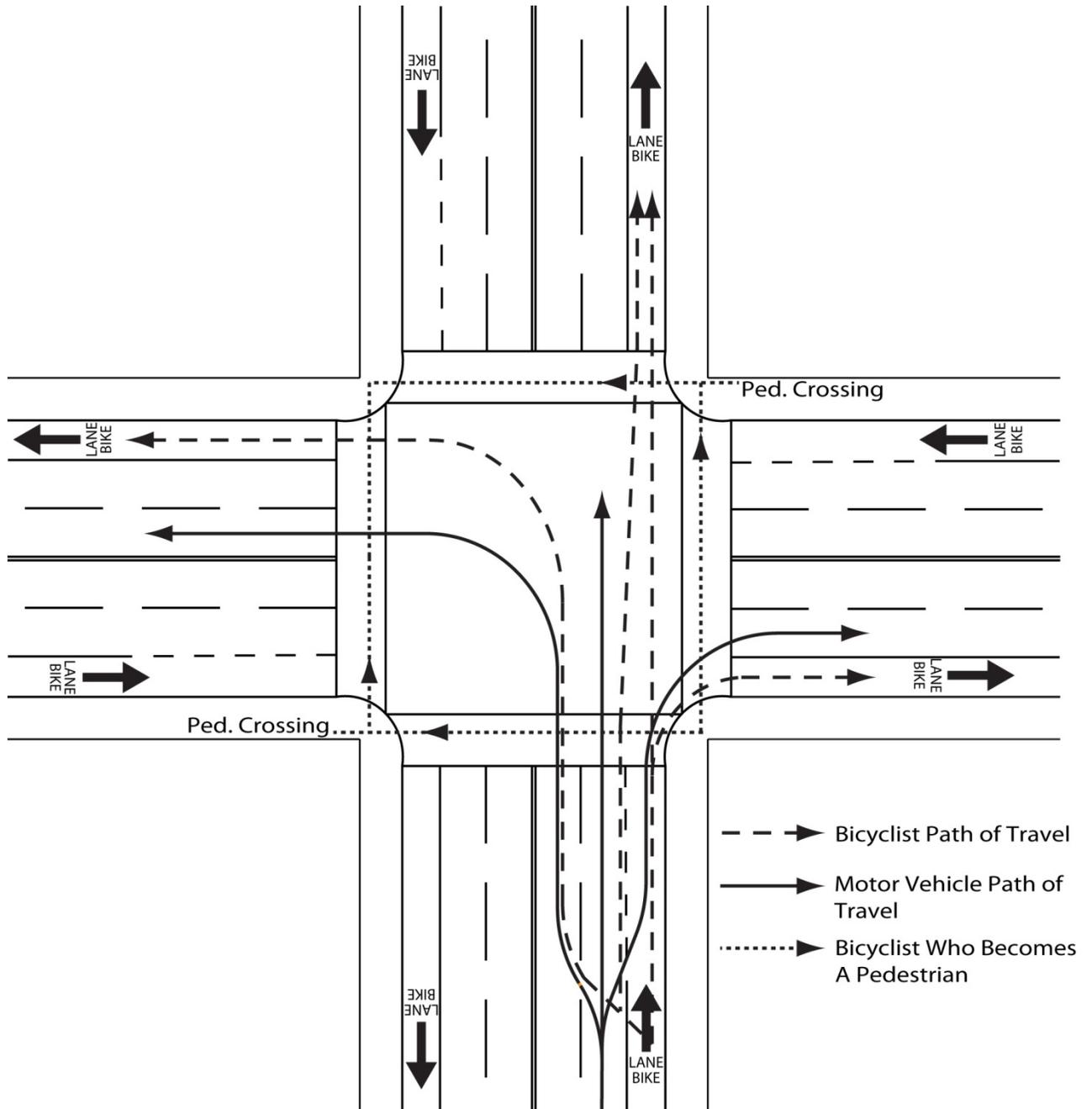
(1) *Treatment of Intersections with Right-Turn-Only Lanes.* Most motor vehicle/bicycle collisions occur at intersections. For this reason, intersection design should be accomplished in a manner that will minimize confusion by motorists and bicyclists, eliminate ambiguity and induce all road users to operate in accordance with the statutory rules of the road in the California Vehicle Code. Right-turn-only lanes should be designed to meet user expectations and reduce conflicts between vehicles and bicyclists.

Figure 403.6A illustrates a typical at-grade intersection of multilane streets without right-turn-only lanes. Bike lanes or shoulders are included on all approaches. Some common movements of motor vehicles and bicycles are shown. A prevalent crash type is between straight-through bicyclists and right-turning motorists, who do not yield to through bicyclists.

Optional right-turn lanes should not be used in combination with right-turn-only lanes on roads where bicycle travel is permitted. The use of optional right-turn lanes in combination with right-turn-only lanes is not recommended in any case where a Class II bike lane is present. This may increase the need for dual or triple right-turn-only lanes, which have

Figure 403.6A

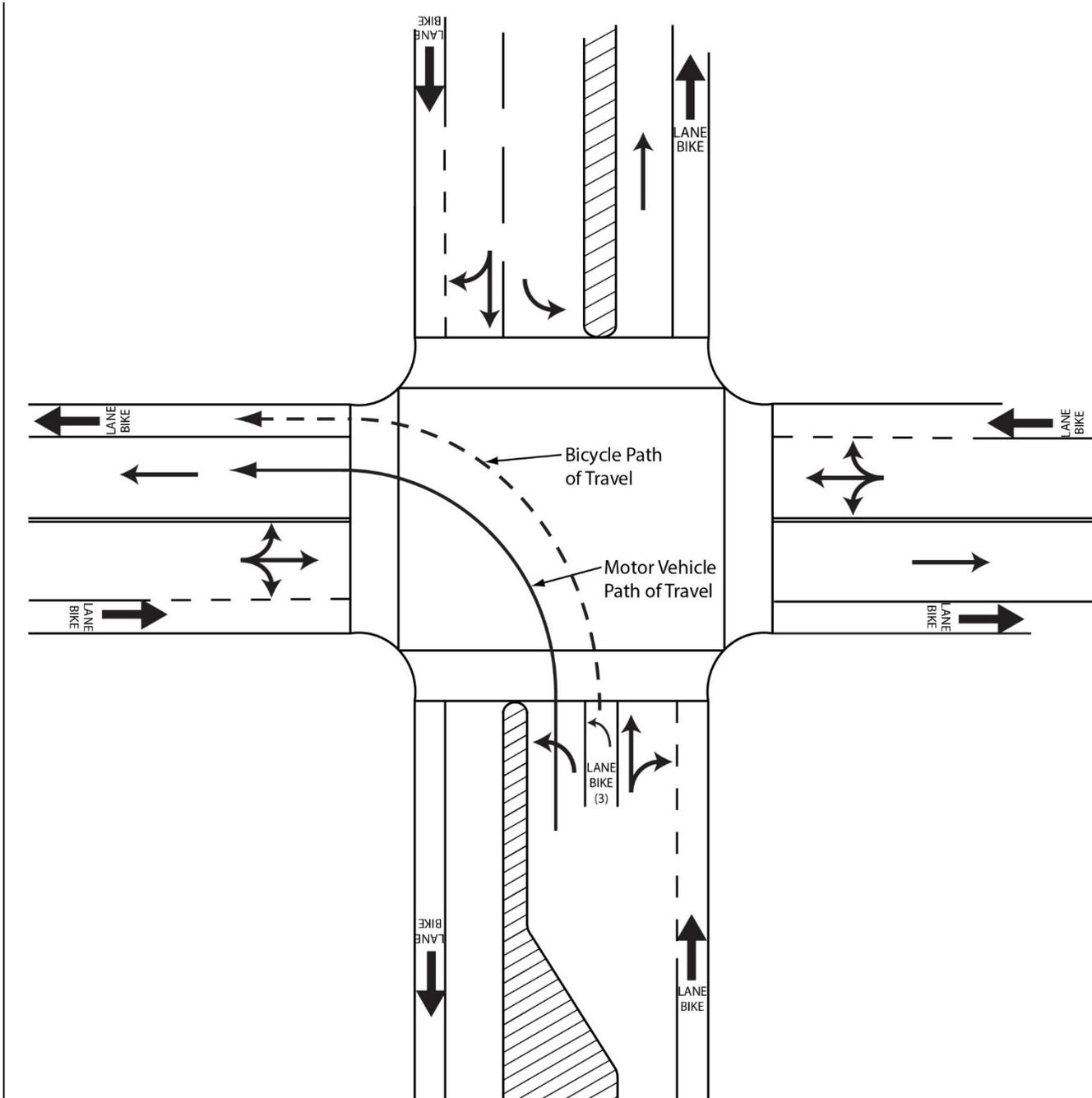
Typical Bicycle and Motor Vehicle Movements at Intersections of Multilane Streets without Right-Turn-Only Lanes



NOTE:

Only one direction is shown for clarity.

Figure 403.6B
Bicycle Left-Turn-Only Lane



NOTES:

- (1) For bicycle lane markings, see the California MUTCD.
- (2) Bicycle detectors are necessary for signalized intersections.
- (3) Left-turn bicycle lane should have receiving bike lane or shoulder.

challenges with visibility between turning vehicles and pedestrians. Multiple right-turn-only lanes should not be free right-turns when there is a pedestrian crossing. If there is a pedestrian crossing on the receiving leg of multiple right-turn-only lanes, the intersection should be controlled by a pedestrian signal head, or geometrically designed such that pedestrians cross only one turning lane at a time.

Locations with right-turn-only lanes should provide a minimum 4-foot width for bicycle use between the right-turn and through lane when bikes are permitted. Configurations that create a weaving area without defined lanes should not be used.

For signing and delineation of bicycle lanes at intersections, consult District Traffic Operations.

Figure 403.6B depicts an intersection with a left-turn-only bicycle lane, which should be considered when bicycle left-turns are common. A left-turn-only bicycle lane may be considered at any intersection and should always be considered as a tool to provide mobility for bicyclists. Signing and delineation options for bicycle left-turn-only lanes are shown in California MUTCD.

- (2) *Design of Intersections at Interchanges.* The design of at-grade intersections at interchanges should be accomplished in a manner that will minimize confusion of motorists, bicyclists, and pedestrians. Higher speed, uncontrolled entries and exits from freeway ramps should not be used at the intersection of the ramps with the local road. The smallest curb return radius should be used that accommodates the design vehicle. Intersections with interior angles close to 90 degrees reduce speeds at conflict points between motorists, bicyclists, and pedestrians. The intersection skew guidance in Index 403.3 applies to all ramp termini at the local road.

403.7 Refuge Areas

Traffic islands should be used to provide refuge areas for bicyclists and pedestrians. See Index 405.4 for further guidance.

403.8 Prohibited Turns

Traffic islands may be used to direct bicycle and motorized vehicle traffic streams in desired directions and prevent undesirable movements. Care should be taken so that islands used for this purpose accommodate convenient and safe pedestrian and bicycle crossings, drainage, and striping options. See Topic 303.

403.9 Effective Signal Control

At intersections with complex turning movements, channelization is required for effective signal control. Channelization permits the sorting of approaching bicycles and motorized vehicles which may move through the intersection during separate signal phases. Pedestrians may also have their own signal phase. This requirement is of particular importance when traffic-actuated signal controls are employed.

The California MUTCD has warrants for the placement of signals to control vehicular, bicycle and pedestrian traffic. Pedestrian activated devices, signals or beacons are not required, but must be evaluated where directional, multilane, pedestrian crossings occur. These locations may include:

- Mid-block street crossings;
- Channelized turn lanes;
- Ramp entries and exits; and
- Roundabouts.

The evaluation, selection, programming and use of a chosen device should be done with guidance from District Traffic Operations.

403.10 Installation of Traffic Control Devices

Channelization may provide locations for the installation of essential traffic control devices, such as "STOP" and directional signs. See Index 405.4 for information about the design of traffic islands.

403.11 Summary

- Give preference to the major move(s).
- Reduce areas of conflict.
- Reduce the duration of conflicts.

- Cross traffic at right angles or skew no more than 75 degrees. (90 degrees preferred.)
- Separate points of conflict.
- Provide speed-change areas and separate turning lanes where appropriate.
- Provide adequate width to shadow turning traffic.
- Restrict undesirable moves with traffic islands.
- Coordinate channelization with effective signal control.
- Install signs in traffic islands when necessary but avoid building conflicts one or more modes of travel.
- Consider all users.

403.12 Other Considerations

- An advantage of curbed islands is they can serve as pedestrian refuge. Where curbing is appropriate, consideration should be given to mountable curbs. See Topic 303 for more guidance.
- Avoid complex intersections that present multiple choices of movement to the motorist and bicyclist.
- Traffic safety should be considered. Collision records provide a valuable guide to the type of channelization needed.

Topic 404 - Design Vehicles

404.1 General

Any vehicle, whether car, bus, truck, or recreational vehicle, while turning a curve, covers a wider path than the width of the vehicle. The outer front tire can generally follow a circular curve, but the inner rear tire will swing in toward the center of the curve.

Some terminology is vital to understanding the engineering concepts related to design vehicles. See Index 62.4 Interchanges and Intersection at Grade for terminology.

404.2 Design Considerations

It may not be necessary to provide for design vehicle turning movements at all intersections along the State route if the design vehicle's route is

restricted or it is not expected to use the cross street frequently. Discuss with Traffic Operation and the local agency before a turning movement is not provided. The goal is to minimize as much as possible conflicts between vehicles, bicycles, pedestrians, and other users of the street, while providing the minimum curb radii appropriate for the given situation. The designer may reference the AASHTO Green Book to select the design vehicle to analyze turning movements to and from the State route. However, turning movements of the State route design vehicle should also be analyzed to determine the impacts from their occasional use.

Both the tracking width and swept width should be considered in the design of roadways for use of the roadway by design vehicles.

Tracking width lines delineate the path of the vehicle tires as the vehicle moves through the turn.

Swept width lines delineate the path of the vehicle body as the vehicle moves through the turn and will therefore always exceed the tracking width. The following list of criteria is to be used to determine whether the roadway can accommodate the design vehicle.

(1) *Traveled way.*

(a) To accommodate turn movements (e.g., at intersections, driveways, alleys, etc.), the travel way width and intersection design should be such that tracking width and swept width lines for the design vehicle do not cross into any portion of the lane for opposing traffic. Encroachment into the shoulder and bike lane is permitted.

(b) Along the portion of roadway where there are no turning options, vehicles are required to stay within the lane lines. **The tracking and swept widths lines for the design vehicle shall stay within the lane as defined in Index 301.1 and Table 504.3A.** This includes no encroachment into Class II bike lanes.

(2) *Shoulders.* Both tracking width and swept width lines may encroach onto paved shoulders to accommodate turning. For design projects where the tracking width lines are shown to encroach onto paved shoulders, the shoulder pavement structure should be engineered to

sustain the weight of the design vehicle. See Index 613 for general traffic loading considerations and Index 626 for tied rigid shoulder guidance. At corners where no sidewalks are provided and pedestrians are using the shoulder, a paved refuge area may be provided outside the swept width of turning vehicle.

- (3) *Curbs and Gutters.* Tires may not mount curbs. If curb and gutter are present and any portion of the gutter pan is likewise encroached, the gutter pan must be engineered to match the adjacent shoulder pavement structure. See Index 613.5(2)(c) for gutter pan design guidance.
- (4) *Edge of Pavement.* To accommodate a turn, the swept width lines may cross the edge of pavement provided there are no obstructions. The tracking width lines shall remain on the pavement structure, including the shoulder, provided that the shoulder is designed to support vehicular traffic. If truck volumes are high, consideration of a wider shoulder is encouraged in order to preserve the pavement edge.
- (5) *Bicycle Lanes.* Where bicycle lanes are considered, the design guidance noted above applies. Vehicles are permitted to cross a bicycle lane to initiate or complete a turning movement or for emergency parking on the shoulder. See the California MUTCD for Class II bike lane markings.

To accommodate turn movements (e.g., intersections, driveways, alleys, etc. are present), both tracking width and swept width lines may cross the broken white painted bicycle lane striping in advance of the right-turn, entering the bicycle lane when clear to do so.

- (6) *Sidewalks.* Tracking width and swept width lines must not encroach onto sidewalks or pedestrian refuge areas, without exception.
- (7) *Obstacles.* Swept width lines may not encroach upon obstacles including, but not limited to, curbs, islands, sign structures, traffic delineators/channelizers, traffic signals, lighting poles, guardrails, trees, cut slopes, and rock outcrops.
- (8) *Appurtenances.* Swept width lines do not include side mirrors or other appurtenances

allowed by the California Vehicle Code, thus, accommodation to non-motorized users of the facility and appurtenances should be considered.

If both the tracking width and swept width lines meet the design guidance listed above, then the geometry is adequate for that design vehicle. Consideration should be given to pedestrian crossing distance, motor vehicle speeds, truck volumes, alignment, bicycle lane width, sight distance, and the presence of on-street parking.

Note that the STAA Design Vehicle has a template with a 56-foot (minimum) and a 67-foot (longer) radius and the California Legal Design Vehicle has a template with 50-foot (minimum) and 60-foot (longer) radii. The longer radius templates are more conservative. The longer radius templates develop less swept width and leave a margin of error for the truck driver. The longer radius templates should be used for conditions where the vehicle may not be required to stop before entering the intersection.

The minimum radius template can be used if the longer radius template does not clear all obstacles. The minimum radius templates demonstrate the tightest turn that the vehicles can navigate, assuming a speed of less than 10 miles per hour.

For offtracking lane width requirements on freeway ramps, see Topic 504.

404.3 Design Tools

District Traffic should be consulted early in the project to ensure compliance with the design vehicle guidance contained in Topic 404. Essentially, two options are available – templates or computer software.

- The turning templates in Figures 404.5A through G are a design aid for determining the swept width and/or tracking width of large vehicles as they maneuver through a turn. The templates can be used as overlays to evaluate the adequacy of the geometric layout of a curve or intersection when reproduced on clear film and scaled to match the highway drawings. These templates assume a vehicle speed of less than 10 miles per hour.
- Computer software such as AutoTURN or AutoTrak can draw the swept width and/or tracking width along any design curve within a CADD drawing program such as MicroStation

or AutoCAD. Dimensions taken from the vehicle diagrams in Figures 404.5A through G may be inputted into the computer program by creating a custom vehicle if the vehicle is not already included in the software library. The software can also create a vehicle turn template that conforms to any degree curve desired.

404.4 Design Vehicles and Related Definitions

(1) *The Surface Transportation Assistance Act of 1982 (STAA).*

(a) STAA Routes. STAA allows certain longer trucks called STAA trucks to operate on the National Network. After STAA was enacted, the Department evaluated State routes for STAA truck access and created Terminal Access and Service Access routes which, together with the National Network, are called the STAA Network. Terminal Access routes allow STAA access to terminals and facilities. Service Access routes allow STAA trucks one-mile access off the National Network, but only at identified exits and only for designated services. Service Access routes are primarily local roads. A “Truck Network Map,” indicating the National Network routes and the Terminal Access routes is posted on the Department’s Office of Truck Services website and is also available in printed form.

(b) STAA Design Vehicle. The STAA vehicle is a truck tractor-semitrailer with the following dimensions: the maximum length of the semitrailer is 48 feet; the kingpin-to-rear-axle (KPR) distance is unlimited by law, although the semitrailer length usually limits this distance to about 43 feet; the maximum body and axle width is 8.5 feet; the tractor length and overall length are unlimited, (Note: a truck tractor is a non-load-carrying vehicle). There is also a STAA double (truck tractor-semitrailer-trailer); however, the double is not used as the design vehicle due to its shorter turning radius. The STAA Design Vehicle is shown in Figures 404.5A and B.

The STAA Design Vehicle in Figures 404.5A or B should be used in the design of all projects on the National Network and on Terminal Access routes. Where use of the STAA Design Vehicle is not practical, the California Legal Design Vehicle shall be used. The STAA design vehicle has a 23-foot wheelbase tractor. The 23-foot wheelbase is an accurate representation of the STAA vehicle tractor on the road today.

(c) STAA Vehicle – 53-Foot Trailer. Another category of vehicle allowed only on STAA routes has a maximum 53-foot trailer, a maximum 40-foot KPR for two or more axles, a maximum 38-foot KPR for a single axle, and unlimited overall length. This vehicle is not to be used as the design vehicle as it is not the worst case for offtracking due to its shorter KPR. The STAA Design Vehicle should be used instead.

(2) *California Legal.*

(a) California Legal Routes. Virtually all State routes off the STAA Network are California Legal routes. There are two types of California Legal routes, the regular California Legal routes and the KPR Advisory Routes. Advisory routes have signs posted that state the maximum KPR length that the route can accommodate without the vehicle offtracking outside the lane. KPR advisories range from 30 feet to 38 feet, in 2-foot increments. California Legal vehicles are allowed to use both types of California Legal routes. California Legal vehicles can also use the STAA Network. However, STAA trucks are not allowed on any California Legal routes. The Truck Network Map indicating the California Legal routes is posted on the Department’s Office of Truck Services website and is also available in printed form.

(b) California Legal Design Vehicle. The California Legal vehicle is a truck tractor-semitrailer with the following dimensions: the maximum overall length is 65 feet; the maximum KPR distance is 40 feet for

semitrailers with two or more axles, and 38 feet for semitrailers with a single axle; the maximum width is 8.5 feet. There are also two categories of California Legal doubles (truck tractor-semi-trailer-trailer); however, the doubles are not used as the design vehicle due to their shorter turning radii. The California Legal Design Vehicle is shown in Figures 404.5C and D.

The California Legal Design Vehicle in Figures 404.5C and D should be used in the design of all non-STAA route interchanges and intersections on California Legal routes and California Legal KPRA Advisory routes for both new construction and rehabilitation projects.

(3) *40-Foot Bus.*

- (a) 40-Foot Bus Routes. All single-unit vehicles, including buses and motor trucks up to 40 feet in length, are allowed on virtually every route in California.
- (b) 40-Foot Bus Design Vehicle. The 40-Foot Bus Design Vehicle shown in Figure 404.5E is an AASHTO standard. Its 25-foot wheelbase and 40-foot length are typical of city transit buses and some intercity buses. At intersections where truck volumes are light or where the predominate truck traffic consists of mostly 3-axle units, the 40-foot bus may be used. Its wheel path sweeps a greater width than 3-axle delivery trucks, as well as smaller buses such as school buses.

(4) *45-Foot Bus & Motorhome.*

- (a) 45-Foot Bus & Motorhome Routes. The “45-foot bus and motorhome” refers to bus and motorhomes over 40 feet in length, up to and including 45 feet in length. These longer buses and motorhomes are allowed in California, but only on certain routes.

The 45-foot tour bus became legal on the National Network in 1991 and later allowed on some State routes in 1995. The 45-foot motorhome became legal in California in 2001, but only on those routes where the 45-foot bus was already allowed. A “Motorcoach and Motorhome

Map” indicating where these longer buses and motorhomes are allowed and where they are not allowed is posted on the Department’s Office of Truck Services website and is also available in printed form. (Note: Motorcoach is a common industry term for tour bus).

- (b) 45-Foot Bus and Motorhome Design Vehicle. The 45-Foot Bus & Motorhome Design Vehicle shown in Figure 404.5F is used by the Caltrans Truck Size Unit for the longest allowable bus and motorhome. Its wheelbase is 28.5 feet. It is also similar to the AASHTO standard 45-foot bus. Typically this should be the smallest design vehicle used on a State highway. It may be used where the State highway intersects local streets without commercial or industrial traffic.

The 45-Foot Bus and Motorhome Design Vehicle shown in Figure 404.5F should be used in the design of all interchanges and intersections on all green routes on the “Motorcoach and Motorhome Map” for both new construction and rehabilitation projects. Check also the longer standard design vehicles on these routes as required – the STAA Design Vehicle and the California Legal Design Vehicle in Indexes 404.3(1) and (2).

(5) *60-Foot Articulated Bus.*

- (a) 60-Foot Articulated Bus Routes. The articulated bus is allowed a length of up to 60 feet per CVC 35400(b)(3)(A). This bus is used primarily by local transit agencies for public transportation. There is no master listing of such routes. Local transit agencies should be contacted to determine possible routes within the proposed project.
- (b) 60-Foot Articulated Bus Design Vehicle. The 60-Foot Articulated Bus Design Vehicle shown in Figure 404.5G is an AASHTO standard. The routes served by these buses should be designed to accommodate the 60-Foot Articulated Bus Design Vehicle.

404.5 Turning Templates & Vehicle Diagrams

Figures 404.5A through G are computer-generated turning templates at an approximate scale of 1"=50' and their associated vehicle diagrams for the design vehicles described in Index 404.3. The radius of the template is measured to the outside front wheel path at the beginning of the curve. Figures 404.5A through G contain the terms defined as follows:

- (1) *Tractor Width* - Width of tractor body.
- (2) *Trailer Width* - Width of semitrailer body.
- (3) *Tractor Track* - Tractor axle width, measured from outside face of tires.
- (4) *Trailer Track* - Semitrailer axle width, measured from outside face of tires.
- (5) *Lock To Lock Time* - The time in seconds that an average driver would take under normal driving conditions to turn the steering wheel of a vehicle from the lock position on one side to the lock position on the other side. The default in AutoTurn software is 6 seconds.
- (6) *Steering Lock Angle* - The maximum angle that the steering wheels can be turned. It is further defined as the average of the maximum angles made by the left and right steering wheels with the longitudinal axis of the vehicle.
- (7) *Articulating Angle* - The maximum angle between the tractor and semitrailer.

Topic 405 - Intersection Design Standards

405.1 Sight Distance

- (1) *Stopping Sight Distance*. See Index 201.1 for minimum stopping sight distance requirements.
- (2) *Corner Sight Distance*.
 - (a) General--At unsignalized intersections a substantially clear line of sight should be maintained between the driver of a vehicle, bicyclist or pedestrian waiting at the crossroad and the driver of an approaching vehicle. Line of sight for all users should be included in right of way, in order to preserve sight lines.

Adequate time must be provided for the waiting user to either cross all lanes of through traffic, cross the near lanes and turn left, or turn right, without requiring through traffic to radically alter their speed.

The values given in Table 405.1A provide 7-1/2 seconds for the driver on the crossroad to complete the necessary maneuver while the approaching vehicle travels at the assumed design speed of the main highway. The 7-1/2 second criterion is normally applied to all lanes of through traffic in order to cover all possible maneuvers by the vehicle at the crossroad. However, by providing the standard corner sight distance to the lane nearest to and farthest from the waiting vehicle, adequate time should be obtained to make the necessary movement. On multilane highways a 7-1/2 second criterion for the outside lane, in both directions of travel, normally will provide increased sight distance to the inside lanes. Consideration should be given to increasing these values on downgrades steeper than 3 percent and longer than 1 mile (see Index 201.3), where there are high truck volumes on the crossroad, or where the skew of the intersection substantially increases the distance traveled by the crossing vehicle.

In determining corner sight distance, a set back distance for the vehicle waiting at the crossroad must be assumed. **Set back for the driver of the vehicle on the crossroad shall be a minimum of 10 feet plus the shoulder width of the major road but not less than 15 feet.** Line of sight for corner sight distance is to be determined from a 3 and 1/2-foot height at the location of the driver of the vehicle on the minor road to a 4 and 1/4-foot object height in the center of the approaching lane of the major road as illustrated in Figure 504.3J. If the major road has a median barrier, a 2-foot object height should be used to determine the median barrier set back.

In some cases the cost to obtain 7-1/2 seconds of corner sight distances

For additional information and guidance, refer to AASHTO, A Policy on Geometric Design of Highways and Streets, the Headquarters Traffic Liaison and the Design Coordinator.

**Table 405.1B
Application of Sight Distance
Requirements**

Intersection Types	Sight Distance		
	Stopping	Corner	Decision
Private Roads	X	X ⁽¹⁾	
Public Streets and Roads	X	X	
Signalized Intersections	X	(2)	
State Route Inter- sections & Route Direction Changes, with or without Signals	X	X	X

NOTES:

- (1) Per Index 405.1(2)(c), the minimum corner sight distance shall be equal to the stopping sight distance as given in Table 201.1. See Index 405.1(2)(a) for setback requirements.
- (2) Apply corner sight distance requirements at signalized intersections whenever possible due to unanticipated violations of the signals or malfunctions of the signals. See Index 405.1(2)(b).

405.2 Left-turn Channelization

- (1) *General.* The purpose of a left-turn lane is to expedite the movement of through traffic by, controlling the movement of turning traffic, increasing the capacity of the intersection, and improving safety characteristics.

The District Traffic Branch normally establishes the need for left-turn lanes.

- (2) *Design Elements.*
 - (a) Lane Width – **The lane width for both single and double left-turn lanes on State highways shall be 12 feet.**

For conventional State highways with posted speeds less than or equal to 40 miles per hour and AADTT (truck

volume) less than 250 per lane that are in urban, city or town centers (rural main streets), the minimum lane width shall be 11 feet.

When considering lane width reductions adjacent to curbed medians, refer to Index 303.5 for guidance on effective roadway width, which may vary depending on drivers' lateral positioning and shy distance from raised curbs.

- (b) Approach Taper -- On conventional highways without a median, an approach taper provides space for a left-turn lane by moving traffic laterally to the right. The approach taper is unnecessary where a median is available for the full width of the left-turn lane. Length of the approach taper is given by the formula on Figures 405.2A, B and C.

Figure 405.2A shows a standard left-turn channelization design in which all widening is to the right of approaching traffic and the deceleration lane (see below) begins at the end of the approach taper. This design should be used in all situations where space is available, usually in rural and semi-rural areas or in urban areas with high traffic speeds and/or volumes.

Figures 405.2B and 405.2C show alternate designs foreshortened with the deceleration lane beginning at the 2/3 point of the approach taper so that part of the deceleration takes place in the through traffic lane. Figure 405.2C is shortened further by widening half (or other appropriate fraction) on each side. These designs may be used in urban areas where constraints exist, speeds are moderate and traffic volumes are relatively low.

- (c) Bay Taper -- A reversing curve along the left edge of the traveled way directs traffic into the left-turn lane. The length of this bay taper should be short to clearly delineate the left-turn move and to discourage through traffic from drifting into the left-turn lane. Table 405.2A gives offset data for design of bay tapers. In urban areas,

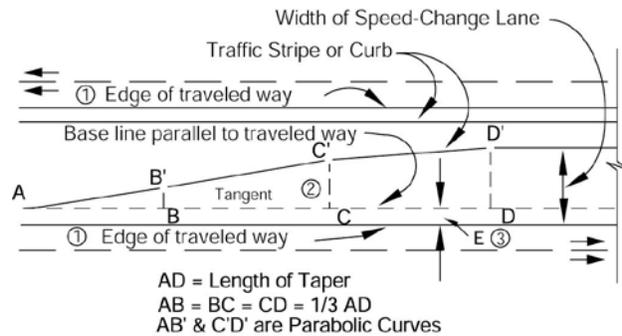
lengths of 60 feet and 90 feet are normally used. Where space is restricted and speeds are low, a 60-foot bay taper is appropriate. On rural high-speed highways, a 120-foot length is considered appropriate.

(d) Deceleration Lane Length -- Design speed of the roadway approaching the intersection should be the basis for determining deceleration lane length. It is desirable that deceleration take place entirely off the through traffic lanes. Deceleration lane lengths are given in Table 405.2B; the bay taper length is included. Where partial deceleration is permitted on the through lanes, as in Figures 405.2B and 405.2C, design speeds in Table 405.2B may be reduced 10 miles per hour to 20 miles per hour for a lower entry speed. In urban areas where cross streets are closely spaced and deceleration lengths cannot be achieved, the District Traffic branch should be consulted for guidance.

(e) Storage Length -- At unsignalized intersections, storage length may be based on the number of turning vehicles likely to arrive in an average 2-minute period during the peak hour. At a minimum, space for 2 vehicles should be provided at 25 feet per vehicle. If the peak hour truck traffic is 10 percent or more, space for at least one passenger car and one truck should be provided. Bus usage may require a longer storage length and should be evaluated if their use is anticipated.

At signalized intersections, the storage length may be based on one and one-half to two times the average number of vehicles that would store per signal cycle depending on cycle length, signal phasing, and arrival and departure rates. At a minimum, storage length should be calculated in the same manner as unsignalized intersection. The District Traffic Branch should be consulted for this information.

**Table 405.2A
Bay Taper for Median
Speed-change Lanes**



LENGTH OF TAPER - feet			OFFSET DISTANCE		
60	90	120	DD' = 10'	DD' = 11'	DD' = 12'
Distance From Point "A"					
-	-	-	0.00	0.00	0.00
5	7.5	10.0	0.16	0.17	0.19
10	15.0	20.0	0.62	0.69	0.75
15	22.5	30.0	1.41	1.55	1.69
B'	20	30.0	2.50	2.75	3.00
	30	45.0	5.00	5.50	6.00
C'	40	60.0	7.50	8.25	9.00
	45	67.5	8.59	9.45	10.31
	50	75.0	9.38	10.31	11.25
	55	82.5	9.84	10.83	11.81
	60	90.0	10.00	11.00	12.00

NOTES:

- (1) The table gives offsets from a base line parallel to the edge of traveled way at intervals measured from point "A". Add "E" for measurements from edge of traveled way.
- (2) Where edge of traveled way is a curve, neither base line nor taper between B & C will be a tangent. Use proportional offsets from B to C.
- (3) The offset "E" is usually 2 ft along edge of traveled way for curbed medians; Use "E" = 0 ft. for striped medians.

**Table 405.2B
Deceleration Lane Length**

Design Speed (mph)	Length to Stop (ft)
30	235
40	315
50	435
60	530

When determining storage length, the end of the left-turn lane is typically placed at least 3 feet, but not more than 30 feet, from the nearest edge of shoulder of the intersecting roadway. Although often set by the placement of a crosswalk line or limit line, the end of the storage lane should always be located so that the appropriate turning template can be accommodated.

- (3) *Double Left-turn Lanes.* At signalized intersections on multilane conventional highways and on multilane ramp terminals, double left-turn lanes should be considered if the left-turn demand is 300 vehicles per hour or more. The lane widths and other design elements of left-turn lanes given under Index 405.2(2) applies to double as well as single left-turn lanes.

The design of double left-turn lanes can be accomplished by adding one or two lanes in the median. See "Guidelines for Reconstruction of Intersections", published by Headquarters, Division of Traffic Operations, for the various treatments of double left-turn lanes.

- (4) *Two-way Left-turn Lane (TWLTL).* The TWLTL consists of a striped lane in the median of an arterial and is devised to address the special capacity and safety problems associated with high-density strip development. It can be used on 2-lane highways as well as multilane highways. Normally, the District Traffic Operations Branch should determine the need for a TWLTL.

The minimum width for a TWLTL shall be 12 feet (see Index 301.1). The preferred width is 14 feet. Wider TWLTL's are occasionally provided to conform with local agency standards. However, TWLTL's wider than 14 feet are not recommended, and in no case should the width of a TWLTL exceed 16 feet. Additional width may encourage drivers in opposite directions to use the TWLTL simultaneously.

405.3 Right-turn Channelization

- (1) *General.* For right-turning traffic, delays are less critical and conflicts less severe than for

left-turning traffic. Nevertheless, right-turn lanes can be justified on the basis of capacity, analysis, and crash experience.

In rural areas a history of high speed rear-end collisions may warrant the addition of a right-turn lane.

In urban areas other factors may contribute to the need such as:

- High volumes of right-turning traffic causing backup and delay on the through lanes.
- Conflicts between crossing pedestrians and right-turning vehicles and bicycles.
- Frequent rear-end and sideswipe collisions involving right-turning vehicles.

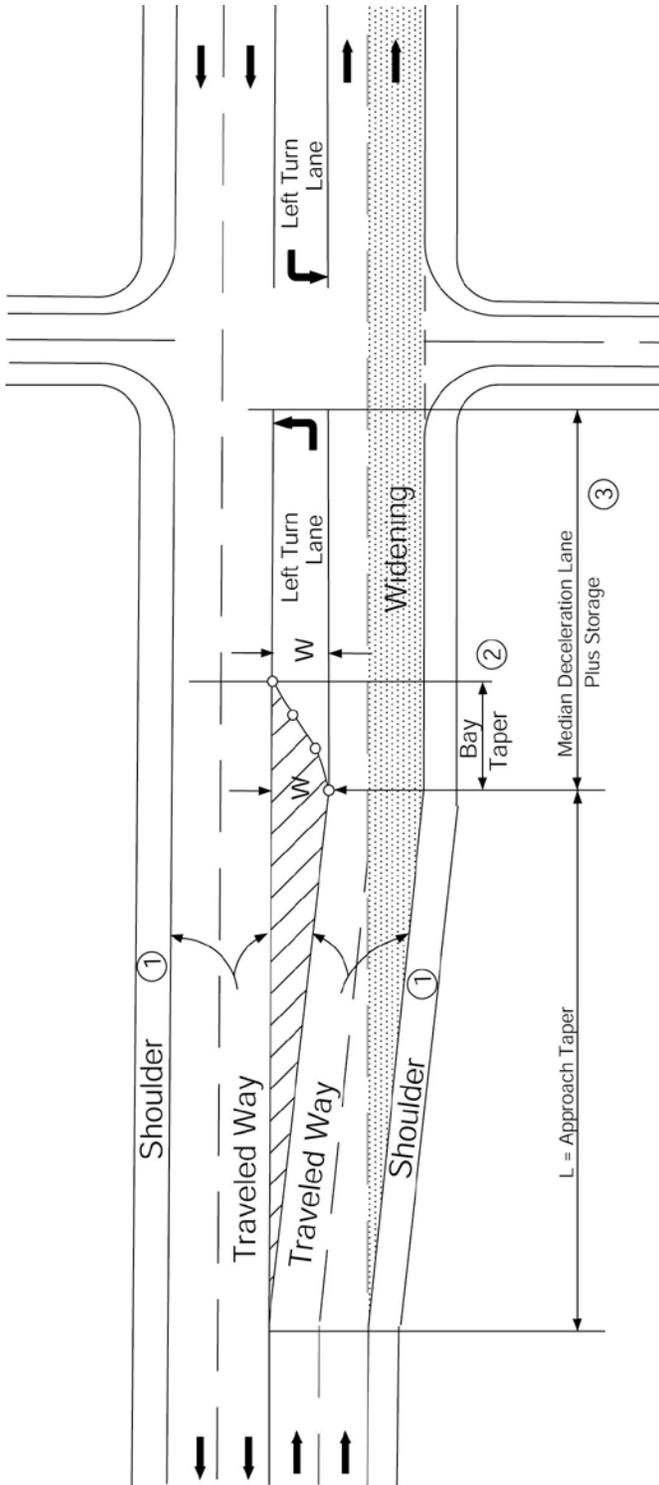
Where right-turn channelization is proposed, lower speed right-turn lanes should be provided to reduce the likelihood of conflicts between vehicles, pedestrians, and bicyclists.

(2) *Design Elements.*

- (a) Lane and Shoulder Width--**Index 301.1 shall be used for right-turn lane width requirements. Shoulder width shall be a minimum of 4 feet.** Although not desirable, lane and shoulder widths less than those given above can be considered for right-turn lanes under the following conditions pursuant to Index 82.2:

- In urban, city or town centers (rural main streets) with posted speeds less than 40 miles per hour in severely constrained situations, if truck or bus use is low, consideration may be given to reducing the right-turn lane width to 10 feet.
- Shoulder widths may also be considered for reduction under constricted situations. Whenever possible, at least a 2-foot shoulder should be provided where the right-turn lane is adjacent to a curb. Entire omission of the shoulder should only be considered in constrained situations and where an 11-foot lane can be constructed.

Figure 405.2A
Standard Left-turn Channelization



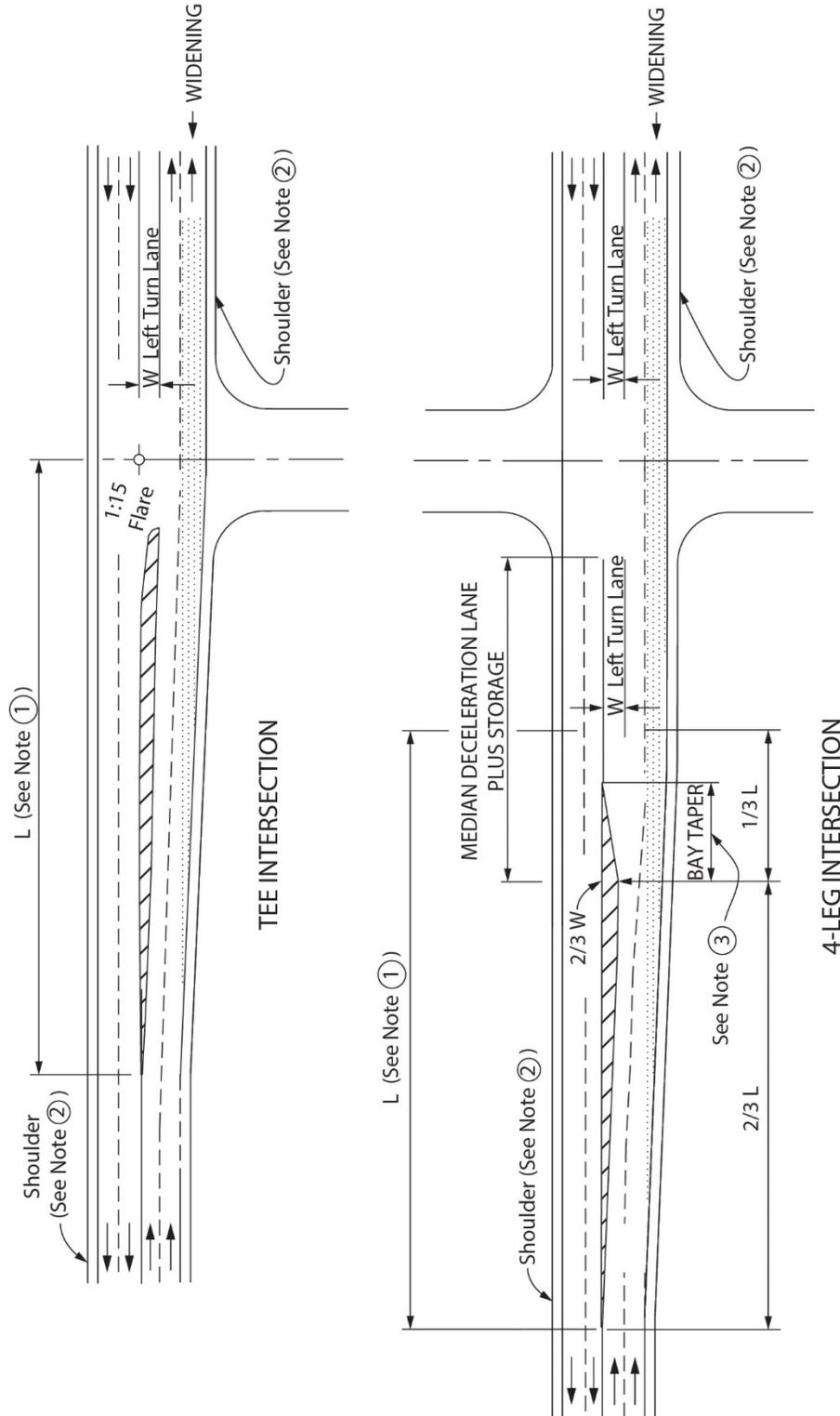
EQUATION: $L = \text{Use } WV, \text{ for } V \geq 45 \text{ mph} \text{ (4)}$
 Or $WV^2/60, \text{ for } V < 45 \text{ mph}$

Where L = Length of Approach Taper - feet
 V = Design Speed - mph
 W = Width of Median Lane - feet

NOTES:

- ① Where width is restricted, shoulder width may be reduced and parking restricted with an approved design exception pursuant to Index 82.2. For bicycle use, a minimum 4-foot shoulder is required (5-foot if gutter is present).
- ② Bay taper length = 60 feet to 120 feet. (See Table 405.2A)
- ③ For deceleration lane length see Table 405.2B.
- ④ Where both sides of roadway are widened, use a fraction of "W" that is proportional to widening on each side.

Figure 405.2B
Minimum Median Left-turn Channelization
(Widening on one Side of Highway)



NOTES:

- ① L = 500 feet Maximum
- ② Where width is restricted, shoulder width may be reduced and parking restricted with an approved design exception pursuant to Index 82.2. For bicycle use, a minimum 4-foot shoulder is required (5-foot if gutter is required)
- ③ Bay Taper Length 60 feet to 120 feet (See Table 405.2A)

EQUATION

Use WW , for $V \geq 45$ mph

$$L = \text{Or } WW^2/60, \text{ for } V < 45 \text{ mph}$$

Where:

L = Length of Transition - feet

W = Width of Median Lane - feet

V = Design Speed - mph

Gutter pans can be included within a shoulder, but cannot be included as part of the travel lane width. Additional right of way for a future right-turn lane should be considered when an intersection is being designed.

- (b) Curve Radius--Where pedestrians are allowed to cross a free right-turning roadway, the curve radius should be such that the operating speed of vehicular traffic is no more than 20 miles per hour at the pedestrian crossing. See NCHRP Report 672, "Roundabouts: An Informational Guide" for guidance on the determination of design speed (fastest path) for turning vehicles. See Index 504.3(3) for additional information.

- (c) Tapers--Approach tapers are usually unnecessary since main line traffic need not be shifted laterally to provide space for the right-turn lane. If, in some rare instances, a lateral shift were needed, the approach taper would use the same formula as for a left-turn lane.

Bay tapers are treated as a mirror image of the left-turn bay taper.

- (d) Deceleration Lane Length--The conditions and principles of left-turn lane deceleration apply to right-turn deceleration. Where full deceleration is desired off the high-speed through lanes, the lengths in Table 405.2B should be used. Where partial deceleration is permitted on the through lanes because of limited right of way or other constraints, average running speeds in Table 405.2B may be reduced 10 miles per hour to 20 miles per hour for a lower entry speed. For example, if the main line speed is 50 miles per hour and a 10 miles per hour deceleration is permitted on the through lanes, the deceleration length may be that required for 40 miles per hour.

- (e) Storage Length--Right-turn storage length is determined in the same manner as left-turn storage length. See Index 405.2(2)(e).

- (3) *Right-turn Lanes at Off-ramp Intersections.* Diamond off-ramps with a free right-turn at the local street and separate right-turn off-ramps

around the outside of a loop will likely cause conflict as traffic volumes increase. Serious conflicts occur when the right-turning vehicle must weave across multiple lanes on the local street in order to turn left at a major cross street close to the ramp terminal. Furthermore, free right-turns create sight distance issues for pedestrians and bicyclists crossing the off-ramp, or pedestrians crossing the local road. Also, rear-end collisions can occur as right-turning drivers slow down or stop waiting for a gap in local street traffic. Free right-turns usually end up with "YIELD", "STOP", or signal controls thus defeating their purpose of increasing intersection capacity.

405.4 Traffic Islands

A traffic island is an area between traffic lanes for channelization of bicycle and vehicle movements or for pedestrian refuge. An island may be defined by paint, raised pavement markers, curbs, pavement edge, or other devices. The California MUTCD should be referenced when considering the placement of traffic islands at signalized and unsignalized locations. For splitter island guidance at roundabouts, see Index 405.10(13).

Traffic islands usually serve more than one function. These functions may be:

- (a) Channelization to confine specific traffic movements into definite channels;
- (b) Divisional to separate traffic moving in the same or opposite direction; and
- (c) Refuge, to aid users crossing the roadway.

Generally, islands should present the least potential conflict to approaching or crossing bicycles and vehicles, and yet perform their intended function.

- (1) *Design of Traffic Islands.* Island sizes and shapes vary from one intersection to another. They should be large enough to command attention. Channelizing islands should not be less than 50 square feet in area, preferably 75 square feet. Curbed, elongated divisional median islands should not be less than 4 feet wide and 20 feet long. All traffic islands placed in the path of a pedestrian crossing must comply with DIB 82. See the Standard Plans for typical island passageway details.

The approach end of each island should be offset 3 feet to the left and 5 feet to the right of approaching traffic, using standard 1:15 parabolic flares, and clearly delineated so that it does not surprise the motorist or bicyclist. These offsets are in addition to the shoulder widths shown in Table 302.1. Table 405.4 gives standard parabolic flares to be used in island design. On curved alignment, parabolic flares may be omitted for small triangular traffic islands whose sides are less than 25 feet long.

The approach nose of a divisional island should be highly visible day and night with appropriate use of signs (reflectorized or illuminated) and object markers. The approach nose should be offset 3 feet from the through traffic to minimize accidental impacts.

- (2) *Delineation of Traffic Islands.* Generally, islands should present the least potential conflict to approaching traffic and yet perform their intended function. See Index 303.2 for appropriate curb type. Islands may be designated as follows:
- (a) Raised paved areas outlined by curbs.
 - (b) Flush paved areas outlined by pavement markings.
 - (c) Unpaved areas (small unpaved areas should be avoided).

On facilities with posted speeds over 40 miles per hour, the use of any type of curb is discouraged. Where curbs are to be used, they should be located at or outside of the shoulder edge, as discussed in Index 303.5.

In rural areas, painted channelization supplemented with raised pavement markers may be more appropriate than a raised curbed channelization. This design is as forgiving as possible and decreases the consequence of a driver's or bicyclist's failure to detect or recognize the curbed island. Consideration for snow removal operations should be determined where appropriate.

In urban areas, posted speeds less than or equal to 40 miles per hour allow more frequent use of curbed islands. Local agency requirements

and matching existing conditions are factors to consider.

(3) *Pedestrian Refuge*

Pedestrian refuge islands allow pedestrians to cross fewer lanes at a time while judging conflicts separately. They also provide a refuge so slower pedestrians can wait for a gap in traffic while reducing total crossing distance.

At unsignalized intersections in rural city/town centers (rural main streets), suburban, or urban areas, a pedestrian refuge should be provided between opposing traffic where pedestrians are allowed to cross 2 or more through traffic lanes in one direction of travel, at marked or unmarked crosswalks. Pedestrian islands at signalized crosswalks should be considered, taking into account crossing distance and pedestrian activity. Note that signalized pedestrian crossings must be timed to allow for pedestrians to cross. See the California MUTCD, Chapter 4E, for further guidance.

Traffic islands used as pedestrian refuge are to be large enough to provide a minimum of 6 feet in the direction of pedestrian travel, without exception.

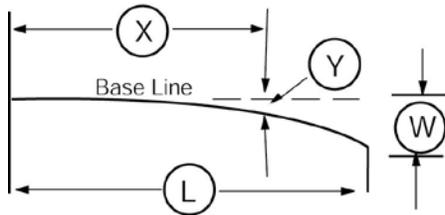
All traffic islands placed in the path of a pedestrian crossing must be accessible, refer to DIB 82 and the Standard Plans for further guidance. An example of a traffic island that serves as a pedestrian refuge is shown on Figure 405.4.

405.5 Median Openings

- (1) *General.* Median openings, sometimes called crossovers, provide for crossings of the median at designated locations. Except for emergency passageways in a median barrier, median openings are not allowed on urban freeways.

Median openings on expressways or divided conventional highways should not be curbed except when the median between openings is curbed, or it is necessary for delineation of traffic signal standards and other necessary hardware, or for protection of pedestrians. In these special cases B4 curbs should be used. An example of a median opening design is shown on Figure 405.5.

Table 405.4
Parabolic Curb Flares Commonly Used



$$Y = \frac{W X^2}{L^2}$$

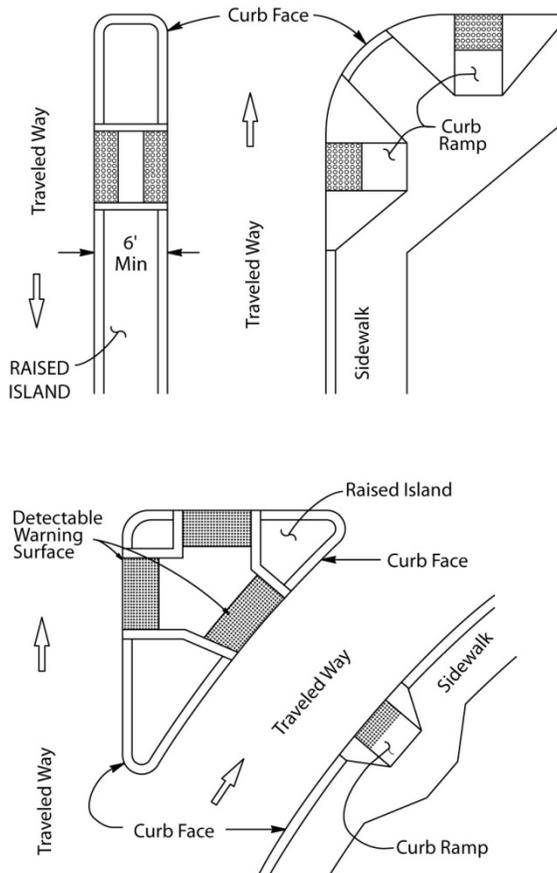
- (L) = Length of flare in feet
- (W) = Maximum offset in feet
- (X) = Distance along base line in feet
- (Y) = Offset from base line in feet

(W) is shown in table thus

OFFSET IN FEET FOR GIVEN "X" DISTANCE																
Distance (L) Length of Flare (X)	10	15	20	25	30	40	45	50	60	70	75	80	90	100	110	120
1:5 FLARES																
25	0.80	1.80	3.20	5.00												
50	0.40		1.60		3.60	6.40		10.00								
1:10 FLARES																
50	0.20		0.80		1.80	3.20		5.00								
100	0.10		0.40		0.90	1.60		2.50	3.60	4.90		6.40	8.10	10.00		
1:15 FLARES																
45	0.15		0.59		1.33	2.37	3.00									
75	0.09		0.36		0.80	1.42		2.22	3.20	4.36	5.00					
90	0.07		0.30		0.67	1.19		1.85	2.67	3.63		4.74	6.00			
120	0.06		0.22		0.50	0.89		1.39	2.00	2.72		3.56	4.50	5.56	6.72	8.00

Figure 405.4

Pedestrian Refuge Island



- (2) *Spacing and Location.* By a combination of interchange ramps and emergency passageways, provisions for access to the opposite side of a freeway may be provided for law enforcement, emergency, and maintenance vehicles to avoid extreme out-of-direction travel. Access should not be more frequent than at three-mile intervals. See Chapter 7 of the Traffic Manual for additional information on the design of emergency passageways.

Emergency passageways should be located only where decision sight distance is available (see Table 201.7).

Median openings at close intervals on other types of highways create conflicts with high speed through traffic. Median openings should be spaced at intervals no closer than 1600 feet.

If a median opening falls within 300 feet of an access opening, it should be placed opposite the access opening.

- (3) *Length of Median Opening.* For any three or four-leg intersection on a divided highway, the length of the median opening should be at least as great as the width of the crossroads pavement, median width, and shoulders. An important factor in designing median openings is the path of the design vehicle making a minimum left turn at 5 miles per hour to 10 miles per hour. The length of median opening varies with width of median and angle of intersecting road.

Usually a median opening of 60 feet is adequate for 90 degree intersections with median widths of 22 feet or greater. When the median width is less than 22 feet, a median opening of 70 feet is needed. When the intersection angle is other than 90 degrees, the length of median opening should be established by using truck turn templates (see Index 404.3).

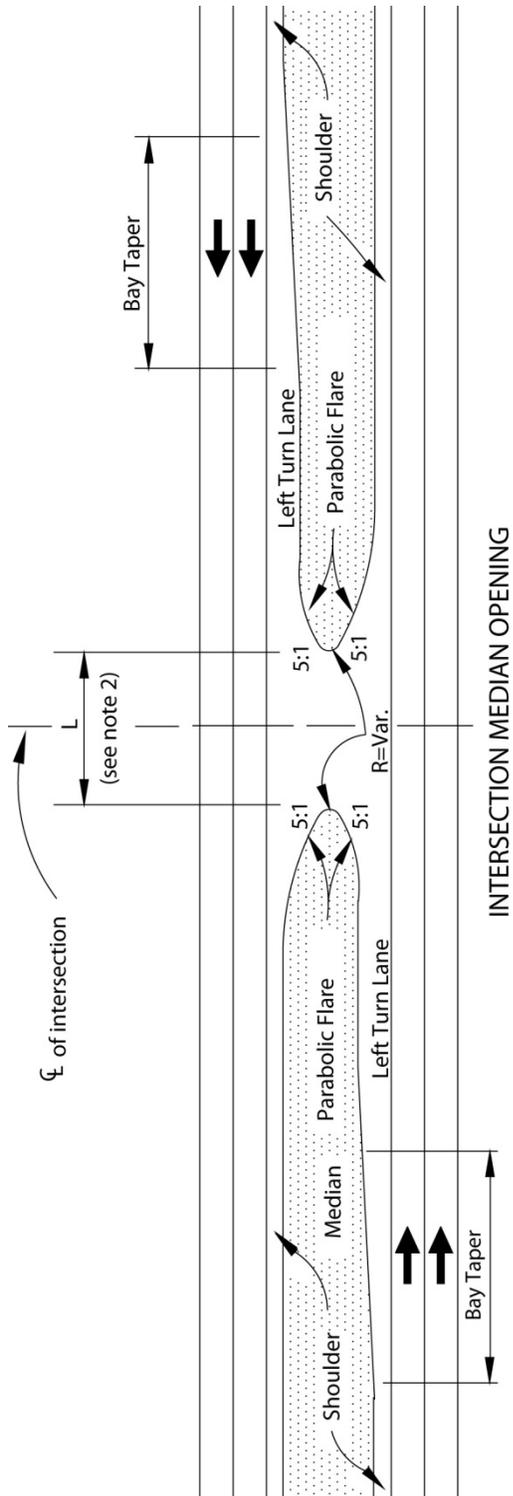
- (4) *Cross Slope.* The cross slope in the median opening should be limited to 5 percent. Crossovers on curves with super elevation exceeding 5 percent should be avoided. This cross slope may be exceeded when an existing 2-lane roadbed is converted to a 4-lane divided highway. The elevation of the new construction should be based on the 5 percent cross slope requirement when the existing roadbed is raised to its ultimate elevation.
- (5) *References.* For information related to the design of intersections and median openings, "A Policy on Geometric Design of Highways and Streets," AASHTO, should be consulted.

405.6 Access Control

The basic guidance which govern the extent to which access rights are to be acquired at interchanges (see Topic 104, Index 205.1 and 504.8 and the PDPM) also apply to intersections at grade on expressways. Cases of access control which frequently occur at intersections are shown in Figure 405.7. This illustration does not presume to cover all situations. Where required by traffic conditions, access should be extended in order to ensure proper operation of the expressway lanes.

Figure 405.5

Typical Design for Median Openings



NOTES:

- ① For length of bay taper, see Table 405.2A.
- ② L = Length of median opening: varies with width of median and angle of intersecting road. Usually for 90° intersection, L = 60 feet for median of 22 feet and wider. L = 70 feet for medians narrower than 22 feet.
- ③ See Index 405.2.
- ④ Pedestrain and bicycle features are not shown on figure.

Reasonable variations which observe the basic principles referred to above are acceptable.

However, negative impacts on the mobility needs of pedestrians, bicyclists, equestrians, and transit users need to be assessed. Pedestrians and bicyclists are sensitive to additional out of direction travel.

405.7 Public Road Intersections

The basic design to be used at right-angle public road intersections on the State Highway System is shown in Figure 405.7. The essential elements are sight distance (see Index 405.1) and the treatment of the right-turn on and off the main highway. Encroachment into opposing traffic lanes by the turning vehicle should be avoided or minimized.

- (1) *Right-turn Onto the Main Highway.* The combination of a circular curve joined by a 2:1 taper on the crossroads and a 75-foot taper on the main highway is designed to fit the wheel paths of the appropriate turning template chosen by the designer.

It is desirable to keep the right-turn as tight as practical, so the “STOP” or “YIELD” sign on the minor leg can be placed close to the intersection.

- (2) *Right-turn Off the Main Highway.* The combination of a circular curve joined by a 150-foot taper on the main highway and a 4:1 taper on the crossroads is designed to fit the wheel paths of the appropriate turning template and to move the rear of the vehicle off the main highway. Deceleration and storage lanes may be provided when necessary (see Index 405.3).

- (3) *Alternate Designs.* Offsets are given in Figure 405.7 for right angle intersections. For skew angles, roadway curvature, and possibly other reasons, variations to the right-angle design are permitted, but the basic rule is still to approximate the wheel paths of the design vehicle.

A three-center curve is an alternate treatment that may be used at the discretion of the designer.

Intersections are major consideration in bicycle path design as well. See Indexes 403.6

and 1003.1(4) for general bicycle path intersection design guidance. Also see Section 5.3 of the AASHTO Guide for the Planning, Design, and Operation of Bicycle Facilities.

405.8 City Street Returns and Corner Radii

The pavement width and corner radius at city street intersections is determined by the type of vehicle to be accommodated and the mobility needs of pedestrians and bicyclists, taking into consideration the amount of available right of way, the types of adjoining land uses, the place types, the roadway width, and the number of lanes on the intersecting street.

At urban intersections, the California truck or the Bus Design Vehicle template may be used to determine the corner radius. Where STAA truck access is allowed, the STAA Design Vehicle template should be used giving consideration to factors mentioned above. See Index 404.3.

Smaller radii of 15 feet to 25 feet are appropriate at minor cross streets where few trucks or buses are turning. Local agency standards may be appropriate in urban and suburban areas.

Encroachment into opposing traffic lanes must be avoided.

405.9 Widening of 2-lane Roads at Signalized Intersections

Two-lane State highways may be widened at intersections to 4-lanes whenever signals are installed. Sometimes it may be necessary to widen the intersecting road. The minimum design is shown in Figure 405.9. More elaborate treatment may be warranted by the volume and pattern of traffic movements. Unusual turning movement patterns may possibly call for a different shape of widening.

The impact on pedestrian and bicycle traffic mobility of larger intersections should be assessed before a decision is made to widen an intersection.

405.10 Roundabouts

Roundabout intersections on the State highway system must be developed and evaluated in accordance with National Cooperative Highway Research Program (NCHRP) Report 672 entitled “Roundabouts: An Informational Guide, 2nd ed.” (NCHRP Guide 2) dated October 2010 and Traffic

Operations Policy Directive (TOPD) Number 13-02. Also see Index 401.5 for general information and guidance. See Figure 405.10 Roundabout Geometric Elements for nomenclature associated with roundabouts. Signs, striping and markings at roundabouts are to comply with the California MUTCD.

A roundabout is a form of circular intersection in which traffic travels counterclockwise around a central island and entering traffic must yield to the circulating traffic. Roundabouts feature, among other things, a central island, a circulatory roadway, and splitter islands on each approach. Roundabouts rely upon two basic and important operating principles:

- (a) Speed reduction at the entry and through the intersection will be achieved through geometric design and,
- (b) The yield-at-entry rule, which requires traffic entering the intersection to yield to traffic that is traveling in the circulatory roadway.

Benefits of roundabouts are:

- Fewer conflict points typically result in fewer collisions with less severity. Over half of vehicle to vehicle points of conflict associated with intersections are eliminated with the use of a roundabout. Additionally, a roundabout separates the points of conflict which eases the ability of the users to identify a conflict and helps prevent conflicts from becoming collisions.
- Roundabouts are designed to reduce the vehicular speeds at intersections. Lower speeds lessens the vehicular collision severity. Likewise, studies indicate that pedestrian and bicyclist collisions with motorized vehicles at lower speeds significantly reduce their severity.
- Roundabouts allow continuous free flow of vehicles and bicycles when no conflicts exist. This results in less noise and air pollution and reduces overall delays at roundabout intersections.

Except as indicated in this Index, the standards elsewhere in this manual do not apply to roundabouts. For the application of design standards, the approach ends of the splitter islands define the boundary of a roundabout intersection,

see Figure 405.10. The design standards elsewhere in this manual apply to the approach legs beyond the approach ends of the splitter islands.

(1) *Design Period.*

The design period guidance provided in Index 103.2 applies to roundabouts. When staging improvements, see NCHRP Guide 2, Section 6.12.

(2) *Design Vehicles* - See Topic 404.

The turning path for the design vehicle, see Index 404.5, dictates many of the roundabout dimensions. The design vehicle tracking and swept width are to be used when designing all the entries and exits, where design vehicles are unrestricted (see Index 404.2), and the circulatory roadway. The percentage of trucks and their lane utilization is an important consideration on multilane roundabouts when determining if the design will allow trucks to stay within their own lane or encroach into the adjacent lane. If permit vehicles larger than the design vehicle occasionally use the proposed roundabout, they can be accommodated by having removable signs or other removable features in the central island or around the circular path to ensure their swept path can negotiate the roundabout. Roundabouts should not be overdesigned for the occasional permit vehicle.

To accurately simulate the design vehicle swept width traveling through a roundabout, the minimum speed of the design vehicle used in computer simulation software (e.g., Auto TURN) should be 10 mph through the roundabout.

(3) *Inscribed Circle Diameter.*

At single lane roundabouts, the size of the inscribed circle is largely dependent upon the turning requirements of the design vehicle. The inscribed circle diameter must be large enough to accommodate: (a) the STAA design vehicle for all roundabouts on the National Network and on Terminal Access routes; and, (b) the California Legal design vehicle on all non-STAA route intersections on California Legal routes and California Legal KPRA Advisory routes, while maintaining adequate

deflection curvature to ensure appropriate travel speeds for smaller vehicles. The design vehicle is to navigate the roundabout with the front tractor wheels off the truck apron, if one is present. Transit vehicles, fire engines and single-unit delivery vehicles are also to be able to navigate the roundabout without using the truck apron, if one is present. The inscribed circle diameter for a single lane roundabout generally ranges between 105 feet to 150 feet to accommodate the California Legal design vehicle and 130 feet to 180 feet to accommodate the STAA design vehicle.

At multilane roundabouts, the inscribed circle diameter is to achieve adequate alignment of the natural vehicle path while maintaining deflection curvature to ensure appropriate travel speeds. To achieve both of these design objectives requires a slightly larger diameter than used for a single lane roundabout. The inscribed circle diameter for a multilane (2-lane) roundabout generally ranges between 150 feet to 220 feet to accommodate the California Legal design vehicle for non-STAA route intersections on California Legal routes and California Legal KPRA Advisory routes, and 165 feet to 220 feet to accommodate the STAA design vehicle for roundabouts on the National Network and on Terminal Access routes. Similar to a single lane roundabout, the design vehicle is to be able to navigate a multilane roundabout with the front tractor wheels staying off the truck apron, if one is present. Transit vehicles, fire engines and single-unit delivery vehicles are also to be able to navigate the roundabout without using the truck apron, if one is present.

(4) *Entry Speeds.*

Lowering the speed of vehicles entering and traveling through the roundabout is a primary design objective that is achieved by approach alignment and entry geometry.

The following entry speeds should not be exceeded:

- Single lane roundabouts, 25 mph.
- Multilane roundabouts, 30 mph.

For fastest path evaluation, see NCHRP Guide 2, Section 6.7.1.

(5) *Exit Design.*

Similar to entry design, exit design flexibility is required to achieve the optimal balance between competing design variables and project objectives to provide adequate capacity and, essentially, safety while minimizing excessive property impacts and costs. Thus, the selection of a curved versus tangential design is to be based upon the balance of each of these criteria. Exit design is influenced by the place type, pedestrian demand, bicyclist needs, the design vehicle and physical constraints. The exit curb radii are usually larger than the entry curb radii in order to minimize the likelihood of congestion and crashes at the exits. However, the desire to minimize congestion at the exits needs to be balanced with the need to maintain an appropriate operating speed through the pedestrian crossing. Therefore, the exit path radius should not be significantly greater than the circulating path radius to ensure low speeds are maintained at the pedestrian crossing.

(6) *Number of Legs Serving the Roundabout.*

Intersections with more than four legs are often difficult to manage operationally. Roundabouts are a proven traffic control device in such situations. However, it is necessary to ensure that the design vehicle can maneuver through all unrestricted legs of the roundabout.

(7) *Pedestrian Use.*

Sidewalks around the circular roadway are to be designed as shared-use paths, see Index 405.10(8)(c). However, the guidance in Design Information Bulletin (DIB) 82 Pedestrian Accessibility Guidelines for Highway Projects must also be followed when designing these shared-use facilities around a roundabout. If there is a difference in the standards, the guidance in DIB 82 is to be followed. In addition,

- (a) Pedestrian curb ramps need to be differentiated from bike ramps:

- The grooved border differentiates a pedestrian curb ramp from a bicycle ramp. Bicycle ramps for the use of bicyclists are not to utilize a grooved border.
 - Detectable warning surface (truncated domes) are required on curb ramps. They are not to be used on a bike ramp.
- (b) Truck aprons and mountable curbs are not to be placed in the pedestrian crossing areas.
- (c) See the California MUTCD for the signs and markings used at roundabouts.
- (8) *Bicyclist Use.*
- (a) General. Bicyclists may choose to travel in the circular roadway of a roundabout by taking a lane, while others may decide to travel using the shared-use path to bypass the circular roadway. Therefore, the approach and circular roadways, as well as the shared-use path all need to be designed for the mobility needs of bicyclists. See the California MUTCD for the signs and markings used at roundabouts.
- (b) Bicyclist Use of the Circular Roadway. Single lane roundabouts do not require bicyclists to change lanes in the circular roadway to select the appropriate lane for their direction of travel, so they tend to be comfortable for bicyclists to use. Even two-lane roundabouts, which may have straighter paths of travel that can lead to faster vehicular traveling speeds, appear to be comfortable for bicyclists that prefer to travel like vehicles. Roundabouts that have more than two circular lanes can create complexities in signing and striping (see the California MUTCD for guidance), and their operating speed may cause some bicyclists to decide to bypass the circular roadway and use the bicycle ramp that provides access to the shared-use path around the roundabout.
- (c) Bicyclists Use of the Shared-Use Path. The shared-use path is to be designed

using the guidance in Index 1003.1 for Class I Bikeways and in NCHRP Guide 2 Section 6.8.2.2. However, the accessibility guidance in DIB 82 must also be followed when designing these shared-use facilities around a roundabout. If there is a difference in the standards, the accessibility guidance in DIB 82 is to be followed to ensure the facility is accessible to pedestrians with disabilities.

Bicycle ramps are to be located to avoid confusion as curb ramps for pedestrians. Also see Index 405.10(7) for guidance on how to differentiate the two types of ramps. The design details and width of the ramp are also important to the bicyclist. Bicyclists approaching the bicycle ramp need to be provided the choice of merging left into the lane or moving right to use the bicycle ramp. Bicycle ramps should be placed at a 35 to 45 degree angle to the departure roadway and the sidewalk to enable the bicyclists to use the ramp and discourage bicyclists from entering the shared-use path at a speed that is detrimental to the pedestrians. The shared-use path should be designated as Class I Bikeways; however, appropriate regulatory signs may need to be posted if the local jurisdiction has a law(s) that prohibit bicyclists from riding on a sidewalk.

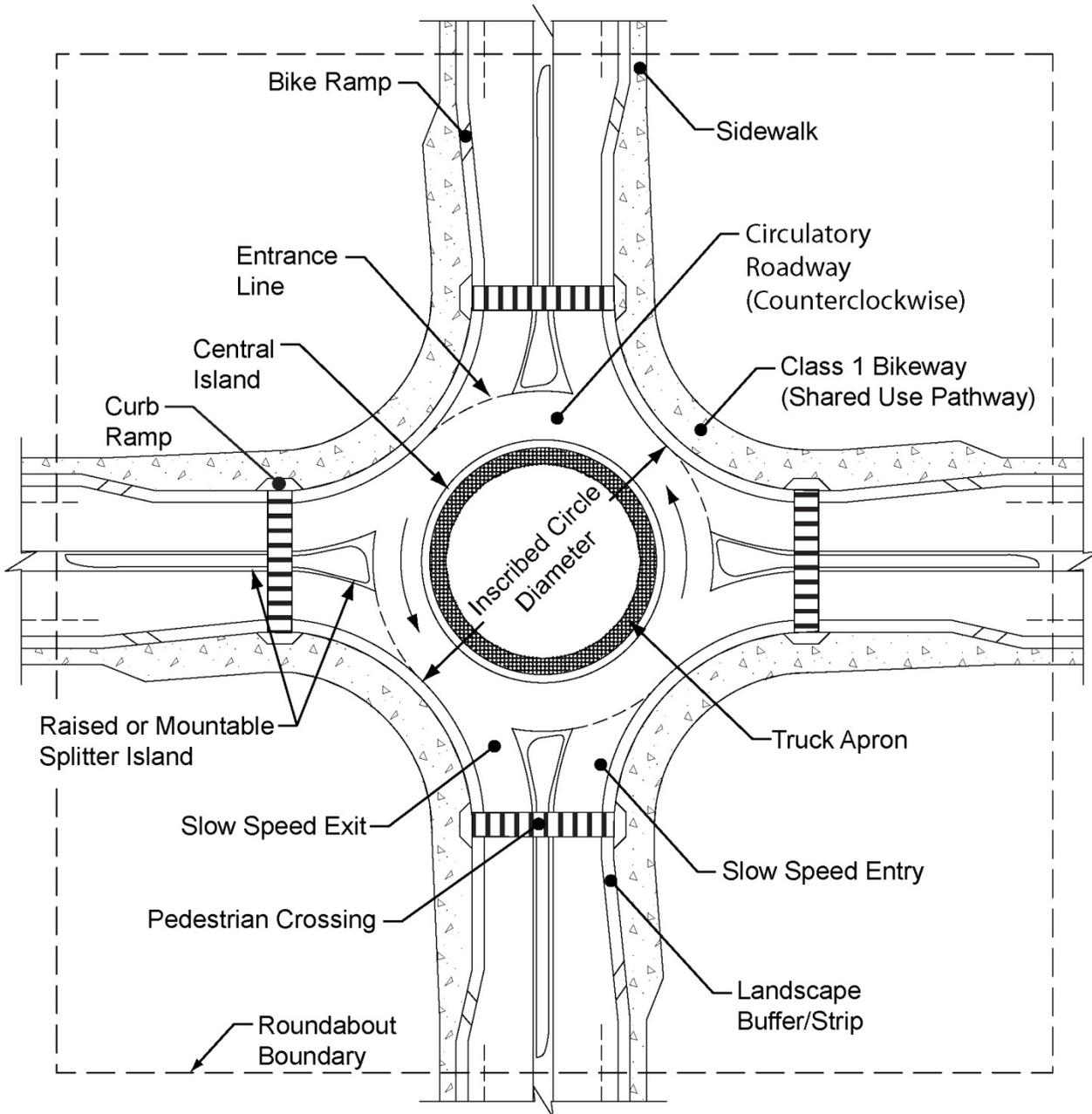
A landscape buffer or strip between the shared-use/Class I Bikeway and the circular roadway of the roundabout is needed and should be a minimum of 2 feet wide.

Pedestrian crossings may also be used by bicyclists; thus, these shared-use crossings need to be designed for both bicyclist and pedestrian needs.

(9) *Transit Use.*

Transit vehicles and buses will not have difficulty negotiating a roundabout when it has been designed using the California Legal design vehicle or the STAA design vehicle. However, to minimize passenger discomfort, a roundabout should be designed such that the

**Figure 405.10
Roundabout Geometric Elements**



NOTE:

This figure is provided to only show nomenclature and is not to be used for design details.

transit vehicle or bus does not use the truck apron, if one is present.

(10) *Stopping Sight Distance and Visibility.*

See Index 201.1 for stopping sight distance guidance at roundabouts.

It is desirable to create a domed or mounded central island, between 3.5 to 6 feet high, to focus attention on the approach and through roundabout alignment. A domed central island provides a visual screen from downstream alignment and other distractions.

(11) *Speed Consistency.*

Consistency in operating speeds between the various movements within the roundabout can minimize collisions between traffic streams. The operating speeds between competing traffic streams and between consecutive geometric elements should be minimized such that the maximum speed differential between them is no more than 15 mph; it is preferred that the operating speed differential be less than 10 mph.

(12) *Path Alignment (Natural Path).*

As two traffic streams approach the roundabout in adjacent lanes, drivers and bicyclists will be guided by lane markings up to the entrance line. At the yield point, they will continue along their natural trajectory into the circulatory roadway. The speed and orientation of the design vehicle at the entrance line determines what can be described as its natural path. The geometry of the exits also affects the natural path that the design vehicle travels. The natural path of two vehicles are not to overlap, see NCHRP Guide 2, Section 6.7.2.

(13) *Splitter Islands.*

Splitter islands (also called separator islands, divisional islands, or median islands) will be provided on all roundabouts. The purpose is to provide refuge for pedestrians, assist in controlling speeds, guide traffic into the roundabout, physically separate entering and exiting traffic streams, and deter wrongway movements.

The total length of the raised island should be at least 50 feet although 100 feet is desirable. On higher speed roadways, splitter island lengths of 150 feet or more is beneficial. Additionally, the splitter island should extend beyond the end of the exit curve to prevent exiting traffic from crossing into the path of approaching traffic. The splitter island width should be a minimum of 6 feet at the pedestrian crossing to adequately provide refuge for pedestrians.

Posted speeds on the approach roadway greater than or equal to 45 mph require the splitter island length, as measured from the inscribed circle diameter, to be 200 feet. In some instances, a longer splitter island may be desirable. Concrete curb is to be provided on the right side of the approach roadway equal to the length of the splitter island from the inscribed circle diameter.

(14) *Access Control.*

The access control standards in Index 504.3(3) and 504.8 apply to roundabouts at interchange ramp intersections. The dimensions shown in Index 504.8 are to be measured from the inscribed circle diameter.

Driveways should not be placed within 100 feet from the inscribed circle diameter.

(15) *Lighting.*

Lighting is required at all roundabouts. See the Traffic Manual Chapter 9 as well as consult with the District Traffic Operations Branch.

(16) *Landscaping.*

Landscaping should be designed such that drivers and bicyclists can observe the signing and shape of the roundabout as they approach, allowing adequate visibility for making decisions within the roundabout. The landscaping of the central island can enhance the intersection by making it a focal point, by promoting lower speeds and by breaking the headlight glare of oncoming vehicles or bicycles. It is desirable to create a domed or mounded central island, between 3.5 to 6 feet high, to increase the visibility of the intersection on the approach. Contact the

District Landscape Architecture Unit to provide technical assistance in designing the roundabout landscaping.

(17) *Vertical Clearance.*

The vertical clearance guidance provided in Index 309.2 applies to roundabouts.

(18) *Drainage Design.*

See Chapter 800 to 890 for further guidance.

Topic 406 - Ramp Intersection Capacity Analysis

The following procedure for ramp intersection analysis may be used to estimate the capacity of any signalized intersection where the phasing is relatively simple. It is useful in analyzing the need for additional turning and through traffic lanes. For a more complete analysis refer to the Highway Capacity Manual.

(a) Ramp Intersection Analysis--For the typical local street interchange there is usually a critical intersection of a ramp and the crossroads that establishes the capacity of the interchange. The capacity of a point where lanes of traffic intersect is 1500 vehicles per hour. This is expressed as intersecting lane vehicles per hour (ILV/hr). Table 406 gives values of ILV/hr for various traffic flow conditions.

If a single-lane approach at a normal intersection has a demand volume of 1000 vph, for example, then the intersecting single-lane approach volume cannot exceed 500 vph without delay.

The three examples that follow illustrate the simplicity of analyzing ramp intersections using this 1500 ILV/hr concept.

(b) Diamond Interchange--The critical intersection of a diamond type interchange must accommodate demands of three conflicting travel paths. As traffic volumes approach capacity, signalization will be needed. For the spread diamond (Figure 406A), basic capacity analysis is made on the assumption that 3-phase signalization is employed. For the tight diamond (Figure 406B), it is assumed that 4-phase signal timing is used.

(c) 2 Quadrant Cloverleaf--Because this interchange design (Figure 406C) permits 2-phase signalization, it will have higher capacities on the approach roadways. The critical intersection is shared two ways instead of three ways as in the diamond case.

Table 406

Vehicle Traffic Flow Conditions at Intersections at Various Levels of Operation

<i>ILV/hr</i>	Description
<i>< 1200:</i>	Stable flow with slight, but acceptable delay. Occasional signal loading may develop. Free midblock operations.
<i>1200-1500:</i>	Unstable flow with considerable delays possible. Some vehicles occasionally wait two or more cycles to pass through the intersection. Continuous backup occurs on some approaches.
<i>1500 (Capacity):</i>	Stop-and-go operation with severe delay and heavy congestion ⁽¹⁾ . Traffic volume is limited by maximum discharge rates of each phase. Continuous backup in varying degrees occurs on all approaches. Where downstream capacity is restrictive, mainline congestion can impede orderly discharge through the intersection.

NOTE:

(1) The amount of congestion depends on how much the ILV/hr value exceeds 1500. Observed flow rates will normally not exceed 1500 ILV/hr, and the excess will be delayed in a queue.

CHAPTER 500 TRAFFIC INTERCHANGES

Topic 501 - General

Index 501.1 - Concepts

A traffic interchange is a combination of ramps and grade separations at the junction of two or more highways for the purpose of reducing or eliminating traffic conflicts, to improve safety, and increase traffic capacity. Crossing conflicts are reduced by grade separations. Turning conflicts are either eliminated or minimized, depending upon the type of interchange design.

501.2 Warrants

All connections to freeways are by traffic interchanges. An interchange or separation may be warranted as part of an expressway (or in special cases at the junction of two non-access controlled highways), to improve safety or eliminate a bottleneck, or where topography does not lend itself to the construction of an intersection.

501.3 Spacing

The minimum interchange spacing shall be one mile in urban areas, two miles outside of urban areas, and two miles between freeway-to-freeway interchanges and other interchanges. The minimum interchange spacing on Interstates outside of urban areas shall be three miles. These minimum distances are measured between centerlines of adjacent intersecting roadways. To improve operations of closely spaced interchanges the use of auxiliary lanes, grade separated ramps, collector-distributor roads, and/or ramp metering may be warranted.

The standards contained within this Index apply to:

- New interchanges.
- Modifications to existing interchanges including access control revisions for new ramps or the relocation/elimination of existing ramps.

- Projects to increase mainline capacity when existing interchanges do not meet interchange spacing requirements.

See Index 504.7 for additional technical requirements related to interchange spacing. Procedures and documentation requirements are provided in PDPM Chapter 27. See the FHWA publication “Interstate System Access Informational Guide.”

Topic 502 - Interchange Types

502.1 General

The selection of an interchange type and its design are influenced by many factors including the following: speed, volume, and composition of traffic to be served (e.g., trucks, vehicles, bicycles, and pedestrians), number of intersecting legs, and arrangement of the local street system (e.g., traffic control devices, topography, right of way controls), local planning, proximity of adjacent interchanges, community impact, and cost.

The cost of a structure is a considerable investment where the life of a structure may be 50 to 100 years, far beyond that of the project traffic study projections. New or significant modifications to interchanges should take into consideration future needs of the system; the ultimate configuration for the freeway and the potential for local land development well beyond the 20-year traffic study. Choose an interchange type that is compatible with or can easily be modified to accommodate the future growth of the system.

Even though interchanges are designed to fit specific conditions and controls, it is desirable that the pattern of interchange ramps along a freeway follow some degree of consistency. It is frequently desirable to rearrange portions of the local street system in connection with freeway construction in

order to affect the most desirable overall plan for mobility and community development.

Interchange types are characterized by the basic shapes of ramps: namely, diamond, loop, directional, hook, or variations of these types. Many interchange designs are combinations of these basic types. Schematic interchange patterns are illustrated in Figure 502.2 and Figure 502.3. These are classified as:(a) Local street interchanges and (b) Freeway-to-freeway interchanges. See AASHTO, A Policy on Geometric Design of Highways and Streets, for additional examples.

502.2 Local Street Interchanges

The Department's philosophy for highway design has evolved over time. DD-64 Complete Streets, DP-22 Context Sensitive Solutions, DP-05 Multimodal Alternatives and other policies and guidance are a result of that evolution in design philosophy. No longer are freeway interchanges designed with only the needs of motorists in mind. Pedestrian and bicycle traffic needs are to be considered along with the motorized traffic. Local road interchanges ramp termini should be perpendicular to the local road. The high speed, shallow angle, ramp termini of the past are problematic for pedestrians and bicyclists to navigate. Vehicle speeds are reduced by the right angle turn, allowing drivers to better respond to bicycle and pedestrian conflicts. For new construction or major reconstruction consideration must be given to orienting ramps at right angles to local streets. For freeways where bicycles are permitted to use the freeway, ramps need to be designed so that bicyclists can exit and enter the freeway without crossing the higher speed ramp traffic. See Index 400 for type, design, and capacity of intersections at the ramp terminus with the local road.

An interchange is expected to have an on- and off-ramp for each direction of travel. If an off-ramp does not have a corresponding on-ramp, that off-ramp would be considered an isolated off-ramp. **Isolated off-ramps or partial interchanges shall not be used because of the potential for wrong-way movements.** In general, interchanges with all ramps connecting with a single cross street are preferred.

At local road interchanges it is preferable to minimize elevation changes on the local road and instead elevate or depress the freeway. Such designs have the least impact on those users most affected by the elevation changes, such as pedestrians and bicyclists.

Class II bikeways designed through interchanges should be accomplished considering the mobility of bicyclists and should be designed in a manner that will minimize confusion by motorists and bicyclists. Designs which allow high speed merges at on- and off-ramps to local streets and conventional highways have a large impact on bicycle and pedestrian mobility and should not be used. Designers should work closely with the Local Agency when designing bicycle facilities through interchanges to ensure that the shoulder width is not reduced through the interchange area. If maintaining a consistent shoulder width is not feasible, the Class II bikeway must end at the previous local road intersection. A solution on how to best provide for bicycle travel to connect both sides of the freeway should be developed in consultation with the Local Agency and community as well as with the consideration of the local bicycle plan.

(a) Diamond Interchange--The simplest form of interchange is the diamond. Diamond interchanges provide a high standard of ramp alignment, direct turning maneuvers at the crossroads, and usually have minimum construction costs. The diamond type is adaptable to a wide range of traffic volumes, as well as the needs of transit, bicyclists, and pedestrians. The capacity is limited by the capacity of the intersection of the ramps at the crossroad. This capacity may be increased by widening the ramps to two or three lanes at the crossroad and by widening the crossroad in the intersection area. Crossroad widening will increase the length of undercrossings and the width of overcrossings, thus adding to the bridge cost. Roundabouts may provide the necessary capacity without expensive crossroad widening between the ramp termini. Ramp intersection capacity analysis is discussed in Topic 406.

The compact diamond (Type L-1) is most adaptable where the freeway is depressed or

section, geometric design and intersection control of ramp termini, location of separation structures, closing of local roads, frontage road construction, bicycle and pedestrian facilities and work on local roads. Particularly close involvement should occur during preparation of the Project Study Report and Project Report (see the Project Development Procedures Manual). Such reviews can be particularly valuable when exceptions to mandatory or advisory design standards are being considered and alternatives are being sought. The geometric features of all interchanges or modifications to existing interchanges must be approved by the Design Coordinator.

Topic 504 - Interchange Design Standards

504.1 General

Topic 504 discusses the standards that pertain to both local service interchanges (various ramp configurations) and freeway-to-freeway connections. The design standards, policies and practices covered in Indexes 504.2, and 504.5 through 504.8 are typically common to both ramp and connector interchange types. Indexes 504.3 and 504.4 separately discuss ramp standards and freeway-to-freeway connector standards, respectively.

504.2 Freeway Entrances and Exits

- (1) **Basic Policy.** All freeway entrances and exits, except for direct connections with median High-Occupancy Vehicle (HOV) lanes, Express Toll lanes or BRT lanes, shall connect to the right of through traffic.
- (2) **Standard Designs.** Design of freeway entrances and exits should conform to the standard designs illustrated in Figure 504.2A-B (single lane), and Figure 504.3L (two-lane entrances and exits) and/or Figure 504.4 (diverging branch connections), as appropriate.

The minimum deceleration length shown on Figure 504.2B shall be provided prior to the first curve beyond the exit nose to assure adequate distance for vehicles to decelerate before entering the curve. The same standard

should apply for the first curve after the exit from a collector-distributor road. The range of minimum "DL" (distance) vs. "R" (radius) is given in the table in Figure 504.2B. Strong consideration should be given to lengthening the "DL" distance given in the table when the subsequent curve is a descending loop or hook ramp, or if the upstream condition is a sustained downgrade (see AASHTO, A Policy on Geometric Design of Highways and Streets, for additional information).

The exit nose shown on Figure 504.2B may be located downstream of the 23-foot dimension; however, the maximum paved width between the mainline and ramp shoulder edges should be 20 feet. Also, see pavement cross slope requirements in Index 504.2(5).

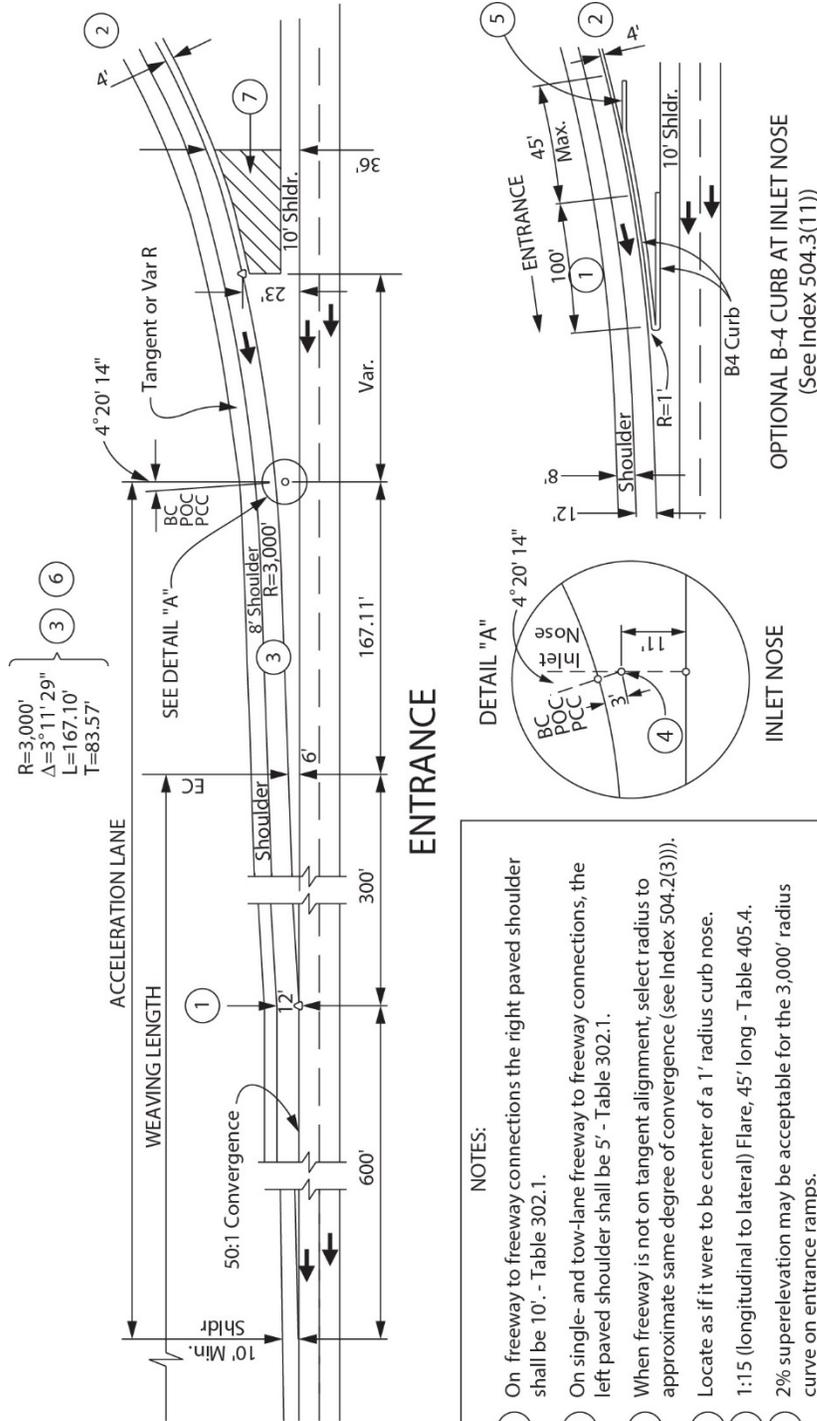
Contrasting surface treatment beyond the gore pavement should be provided on both entrance and exit ramps as shown on Figures 504.2A, 504.2B, and 504.3L. This treatment can both enhance aesthetics and minimize maintenance efforts. It should be designed so that a driver will be able to identify and differentiate the contrasting surface treatment from the pavement areas that are intended for regular or occasional vehicular use (e.g., traveled way, shoulders, paved gore, etc.).

Consult with the District Landscape Architect, District Materials Engineer, and District Maintenance Engineer to determine the appropriate contrasting surface treatment of the facility at a specific location.

Refer to the HOV Guidelines for additional information specific to direct connections to HOV lanes.

- (3) **Location on a Curve.** Freeway entrances and exits should be located on tangent sections wherever possible in order to provide maximum sight distance and optimum traffic operation. Where curve locations are necessary, the ramp entrance and exit tapers should be curved also. The radius of the exit taper should be about the same as the freeway edge of traveled way in order to develop the same degree of divergence as the standard design (see Figure 504.2C).

Figure 504.2A
Single Lane Freeway Entrance



- NOTES:**
- 1 On freeway to freeway connections the right paved shoulder shall be 10' - Table 302.1.
 - 2 On single- and tow-lane freeway to freeway connections, the left paved shoulder shall be 5' - Table 302.1.
 - 3 When freeway is not on tangent alignment, select radius to approximate same degree of convergence (see Index 504.2(3)).
 - 4 Locate as if it were to be center of a 1' radius curb nose.
 - 5 1:15 (longitudinal to lateral) Flare, 45' long - Table 405.4.
 - 6 2% superelevation may be acceptable for the 3,000' radius curve on entrance ramps.
 - 7 Contrasting surface treatment (See Index 504.2(2)). (Advisory Standard)
 - 8 See Index 504.2(6) for pedestrian and bicycle ramp crossings on freeways where bicycle or pedestrian travel is not prohibited. See Index 302.1 for shoulder width standards.

crossroads open to view should be greater than the product of the prevailing speed of vehicles on the crossroads, and the time required for a stopped vehicle on the ramp to execute a left-turn maneuver. This time is estimated to be 7½ seconds.

When proposing uncontrolled entries and exits from freeway ramps with local roads, see the Design of Intersections at Interchanges guidance in Index 403.6(2).

Horizontal sight restrictions may be caused by bridge railings, bridge piers, or slopes. Sight distance is measured between the center of the outside lane approaching the ramp and the eye of the driver of the ramp vehicle assumed 8 feet back from the edge of shoulder at the crossroads. Figure 504.3J illustrates the determination of ramp setback from an overcrossing structure on the basis of sight distance controlled by the bridge rail. The same relationship exists for sight distance controlled by bridge piers or slopes.

Where ramp set back for the 7½ second criterion is unobtainable, sight distance should be provided by flaring the end of the overcrossing structures or setting back the piers or end slopes of an undercrossing structure.

If signals are warranted within 5 years of construction, consideration may be given to installing signals initially in lieu of providing horizontal sight distance which meets the 7½ second criterion. See Part 4 of the California MUTCD, 4B.107(CA). However, this is not desirable and corner sight distance commensurate with design speed should be provided where obtainable (see AASHTO, A Policy on Geometric Design of Highways and Streets).

For additional information on sight distance requirements at signalized intersections, see Index 405.1.

The minimum distance (curb return to curb return) between ramp intersections and local road intersections shall be 400 feet.

The preferred minimum distance should be 500 feet. This does not apply to Resurfacing, Restoration and Rehabilitation (3R), ramp

widening, restriping or other projects which do not reconfigure the interchange. This standard does apply to projects proposing to realign a local street.

Where intersections are closely spaced, traffic operations are often inhibited by short weave distance, storage lengths, and signal phasing. In addition it is difficult to provide proper signing and delineation. The District Traffic Branch should be consulted regarding traffic engineering studies needed to determine the appropriate signage, delineation, and form of intersection control.

- (4) *Superelevation for Ramps.* The factors controlling superelevation rates discussed in Topic 202 apply also to ramps. As indicated in Table 202.2 use the 12 percent e_{max} rate except where snow and ice conditions prevail. In restrictive cases where the length of curve is too short to develop standard superelevation, the highest obtainable rate should be used (see Index 202.5). If feasible, the curve radius can be increased to reduce the standard superelevation rate. Both edge of traveled way and edge of shoulder should be examined at ramp junctions to assure a smooth transition.

Under certain restrictive conditions the standard superelevation rate discussed above may not be required on the curve nearest the ramp intersection of a ramp. The specific conditions under which lower superelevation rates would be considered must be evaluated on a case-by-case basis and must be discussed with the Design Coordinator and documentation as required by the Design Coordinator.

- (5) *Single-lane Ramps.* Single lane ramps are those ramps that either enter into or exit from the freeway as a single lane. These ramps are often widened near the ramp intersection with the crossroads to accommodate turning movements onto or from the ramp. When additional lanes are provided near an entrance ramp intersection, the lane drop should be accomplished over a distance equal to WV. The lane to be dropped should be on the right so that traffic merges left.

Exit ramps in metropolitan areas may require multiple lanes at the intersection with the crossroads to provide additional storage and capacity. If the length of a single lane ramp exceeds 1,000 feet, an additional lane should be provided on the ramp to permit passing maneuvers. Figure 504.3K illustrates alternative ways of transitioning a single lane exit ramp to two lanes. The decision to use Alternate A or Alternate B is generally based on providing the additional lane for the minor movement.

- (6) *Two-lane Exit Ramps.* Where design year estimated volumes exceed 1500 equivalent passenger cars per hour, a 2-lane ramp should be provided.

Provisions should be made for possible widening to three or more lanes at the crossroads intersection. Figure 504.3L illustrates the standard design for a 2-lane exit. An auxiliary lane approximately 1,300 feet long should be provided in advance of a 2-lane exit. For volumes less than 1500 but more than 900, a one-lane width exit ramp should be provided with provision for adding an auxiliary lane and an additional lane on the ramp.

- (7) *Two-lane Entrance Ramps.* These ramps are discouraged in congested corridors. Early discussion with the HQ Traffic Liaison and Design Coordinator or Design Reviewer is recommended whenever two-lane entrance ramps are being considered.

- (8) *Loop Ramps.* Normally, loop ramps should have one lane and shoulders unless a second lane is needed for capacity or ramp metering purposes. Consideration should be given to providing a directional ramp when loop volumes exceed 1500 vehicles per hour. If two lanes are provided, normally only the right lane needs to be widened for trucks. See Topic 404 for additional discussion on lane widths and design of ramp intersections to accommodate the design vehicle. See Index 504.3(1) for a discussion regarding on-ramp widening for trucks.

Radii for loop ramps should normally range from 150 feet to 200 feet. Increasing the radii beyond 200 feet is typically not cost effective

as the slight increase in design speed is usually outweighed by the increased right of way requirements and the increased travel distance. Curve radii of less than 120 feet should also be avoided. Extremely tight curves lead to increased off-tracking by trucks and increase the potential for vehicles to enter the curve with excessive speed. Therefore, consider providing the ramp lane pavement structure on shoulders for curves with a radius less than 300 feet (see Indexes 626.1 and 636.1).

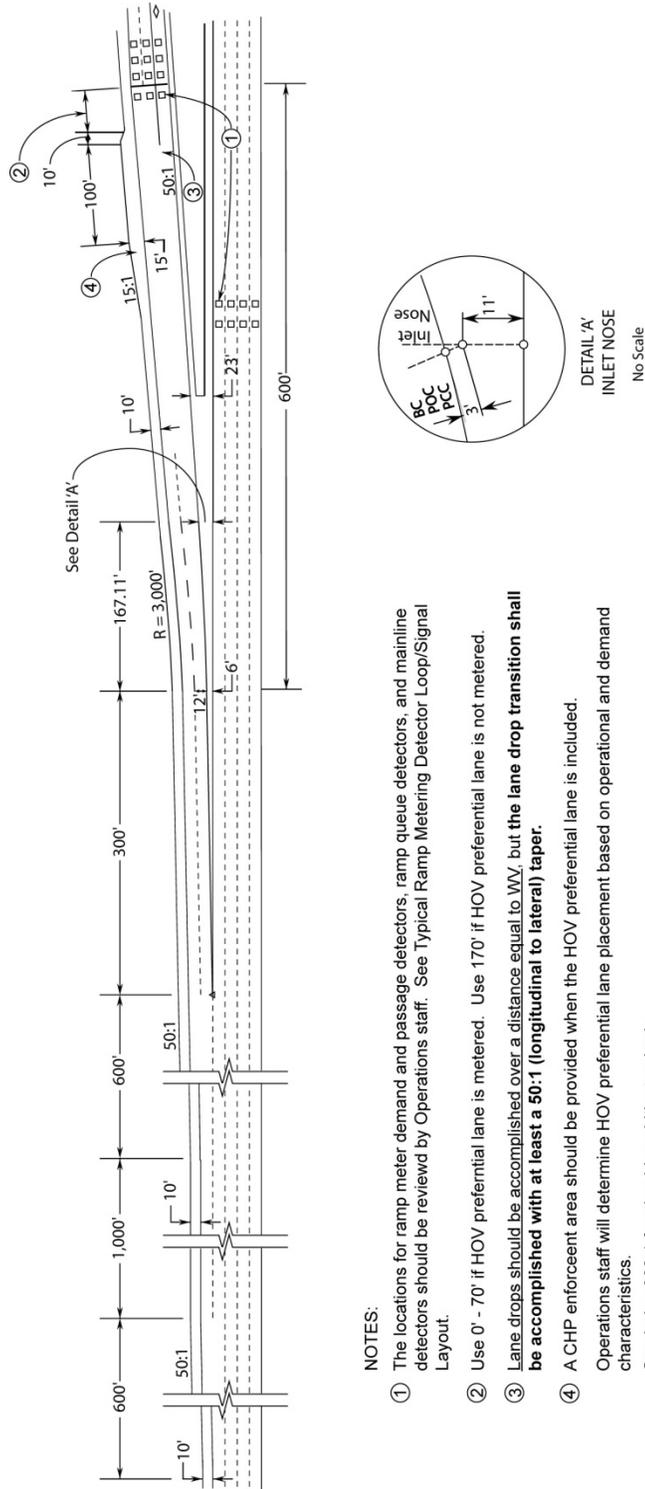
Of particular concern in the design of loop ramps are the constraints imposed on large trucks. Research indicates that trucks often enter loops with excessive speed, either due to inadequate deceleration on exit ramps or due to driver efforts to maintain speed on entrance ramps to facilitate acceleration and merging. Where the loop is of short radius and is also on a steep descent (over 6 percent), it is important to develop the standard 2/3 full superelevation rate by the beginning of the curve (see Index 504.2(5)). When accommodating design vehicles in Rural Developing Corridors that are largely composed of industrial, commercial or retail buildings located separately from housing, the following considerations may be necessary to meet the standard 2/3 full superelevation rate on loop entrance ramps:

- Begin the ramp with a short tangent (75 feet to 100 feet) that diverges from the cross street at an angle of 4 to 9 degrees.
- Provide additional tangent length as site conditions allow.

The Angle of Intersection guidance in Index 403.3 applies to all on-ramps including loops.

- (9) *Distance Between Successive On-ramps.* The minimum distance between two successive on-ramps to a freeway lane should be the distance needed to provide the standard on-ramp acceleration taper shown on Figure 504.2A. This distance should be about 1,000 feet unless the upstream ramp adds an auxiliary lane in which case the downstream ramp should merge with the auxiliary lane in a standard 50:1 (longitudinal to lateral) convergence. The distance between on-ramp noses will then be controlled by interchange geometry.

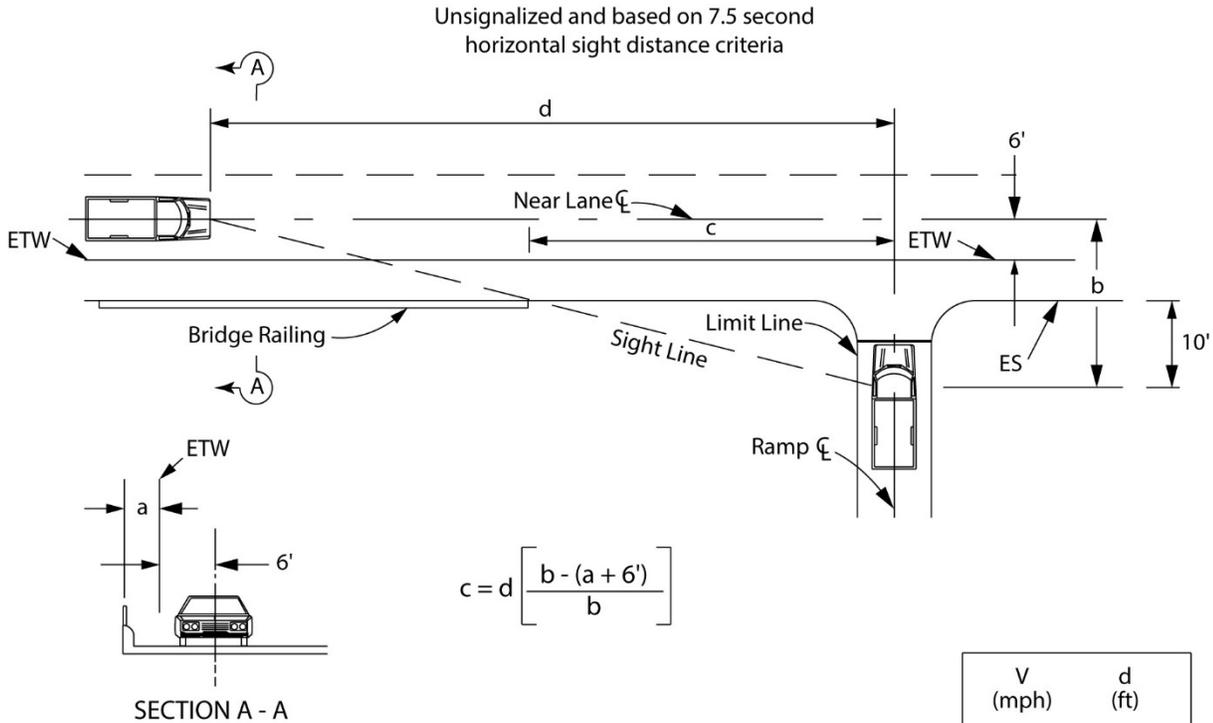
Figure 504.3I
Typical Freeway Connector
3-Lane Meter
(2 mixed-flow lanes + HOV preferential lane)



NOTES:

- ① The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations staff. See Typical Ramp Metering Detector Loop/Signal Layout.
- ② Use 0' - 70' if HOV preferential lane is metered. Use 170' if HOV preferential lane is not metered.
- ③ Lane drops should be accomplished over a distance equal to WV, but the lane drop transition shall be accomplished with at least a 50:1 (longitudinal to lateral) taper.
- ④ A CHP enforcement area should be provided when the HOV preferential lane is included. Operations staff will determine HOV preferential lane placement based on operational and demand characteristics. See Index 302.1 for shoulder width standards. See the California MUTCD for signing and striping typical.

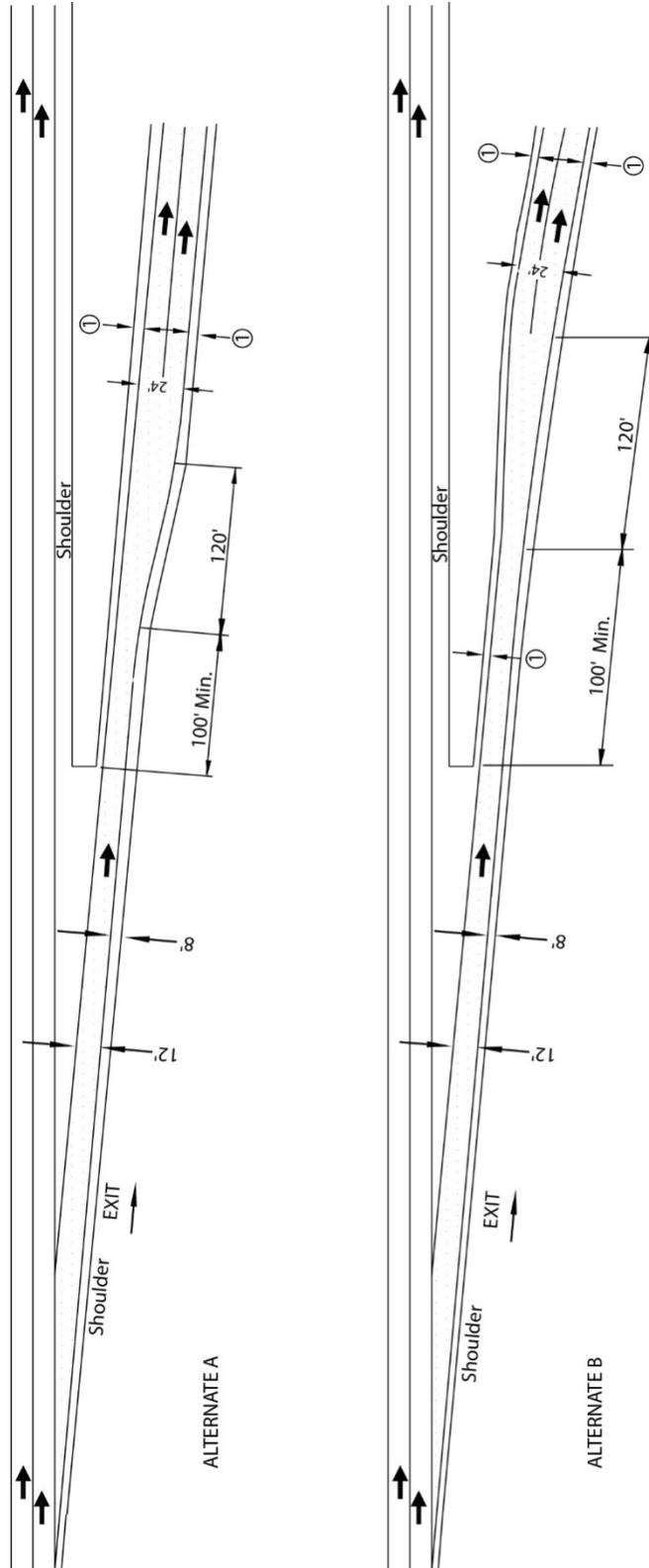
Figure 504.3J
Location of Ramp Intersections
on the Crossroads



- a = Distance from edge of traveled way to bridge railing.
- b = Distance from center of near lane to eye of ramp vehicle driver.
Ramp driver's eye is assumed to be located 10' from the edge of shoulder, but not less than 15' from the ETW (therefore, b = 6' + shoulder width + 10'). See Index 405.1.
- c = Ramp set back from end of bridge railing.
- d = Corner Sight distance along highway from intersection. (See Table above)
Sight distance is measured from a 3½' eye height on the ramp to a 4¼' object height on the crossroad.
- V = Anticipated prevailing speed on crossroad.

V (mph)	d (ft)
25	275
30	330
35	385
40	440
45	495
50	550
55	605
60	660
65	715
70	770

Figure 504.3K
Transition to Two-lane Exit Ramp

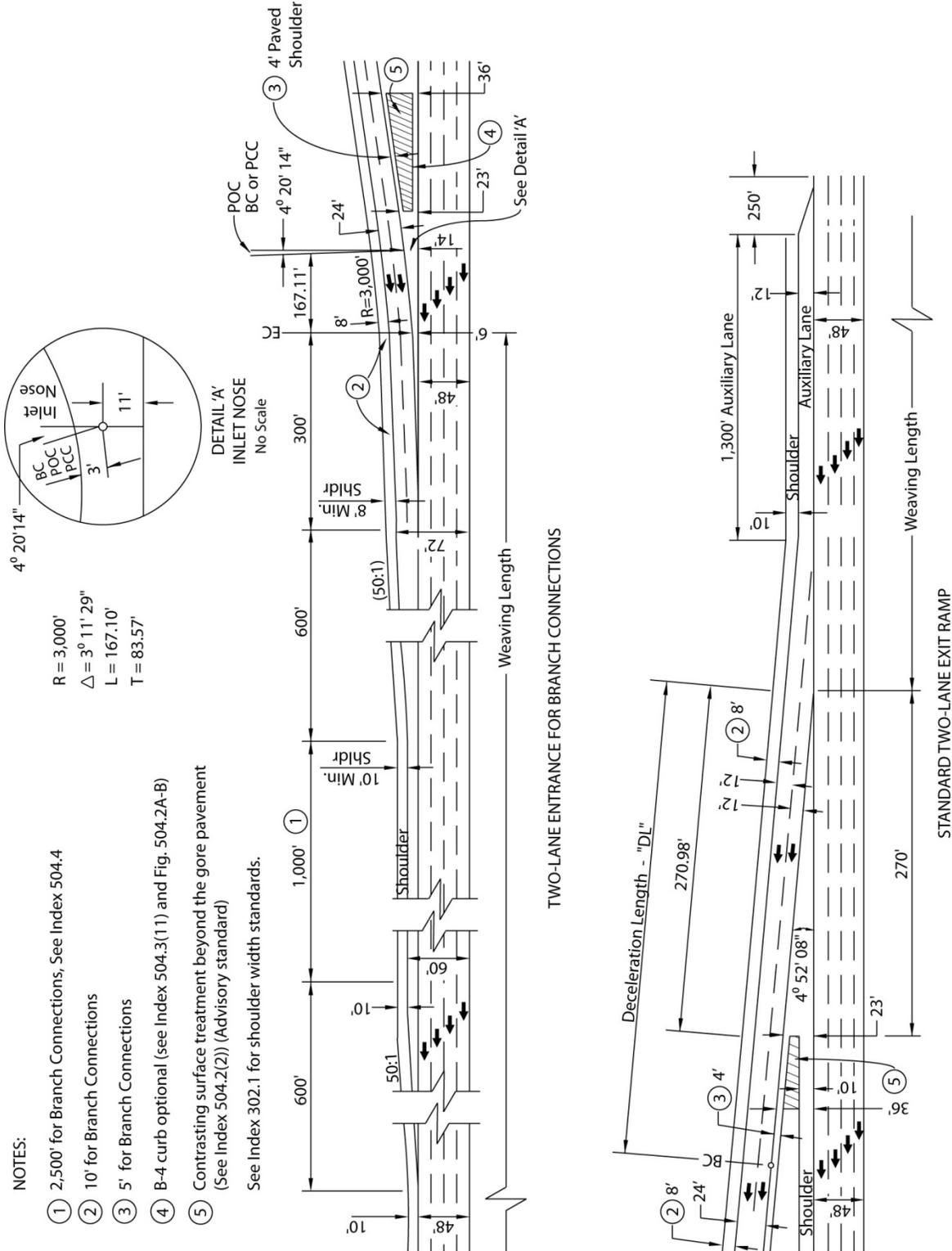


NOTES:

① See Index 302.1 for shoulder width standards. If shoulder reductions occur, see Index 206.3(4) for transitions.

Figure 504.3L

Two-Lane Connectors and Exit Ramps



- (10) *Distance Between Successive Exits.* The minimum distance between successive exit ramps for guide signing should be 1,000 feet on the freeway and 600 feet on collector-distributor roads.
- (11) *Curbs.* Curbs should not be used on-ramps except in the following locations:
- A Type D curb or 4-inch Type B curb (see Index 303.2) may be used on both sides of the separation between freeway lanes and a parallel collector-distributor road.
 - A B4 curb may be used as shown in Figure 504.2A to control drainage or where the gore cross slope would be greater than allowed in Index 504.2(5). When the optional B4 curb is used at the entrance ramp inlet nose, the shoulder adjacent to the curb should be the same width as the ramp shoulder approaching the curb. The B4 gutter pan can be included as part of the shoulder width. As stated in Index 405.4(2), curbs are typically discouraged where posted speeds are over 40 miles per hour. The appropriateness of curbs at gore areas must be determined on a case-by-case basis.
 - Curbs may be used where necessary at the ramp connection with the local street for the protection of pedestrians, for channelization, and to provide compatibility with the local facility.
 - The Type E curb may be used only in special drainage situations, for example, where drainage parallels and flows against the face of a retaining wall.

In general, curbs should not be used on the high side of ramps or in off-ramp gore areas except at collector-distributor roads. The off-tracking of trucks should be analyzed when considering curbs on ramps.

- (12) *Dikes.* Dikes may be used where necessary to control drainage. For additional information see Index 303.3.

504.4 Freeway-to-Freeway Connections

- General.* All of the design criteria discussed in Indexes 501.3, 504.2 and 504.3 apply to freeway to freeway connectors, except as discussed or modified below.
- Design Speed.* The design speed for single lane directional and all branch connections should be a minimum of 50 miles per hour. When smaller radius curves, with lower design speeds, are used the vertical sight distance should be consistent with approaching vehicle speeds. Design speed for loop connectors should be consistent with the radii guidance discussed in Index 504.3(8).
- Grades.* The maximum profile grade on freeway-to-freeway connections should not exceed 6 percent. Flatter grades and longer vertical curves than those used on ramps are needed to obtain increased stopping sight distance for higher design speeds.
- Shoulder Width.*
 - Single-lane and Two-lane Connections--**The width of shoulders on single-lane and two-lane (except as described below) freeway-to-freeway connectors shall be 5 feet on the left and 10 feet on the right. A single lane freeway-to-freeway connector that has been widened to two lanes solely to provide passing opportunities and not due to capacity requirements shall have a 5-foot left shoulder and at least a 5-foot right shoulder** (see Index 504.4(5)).
 - Three-lane Connections--**The width of shoulders on three-lane connectors shall be 10 feet on both the left and right sides.**
- Single-lane Connections.* Freeway-to-freeway connectors may be single lane or multilane. Where design year volume is between 900 and 1500 equivalent passenger cars per hour, initial construction should provide a single lane connection with the capability of adding an additional lane. Single lane directional connectors should be designed using the general configurations shown on Figure

504.2A and 504.2B, but utilizing the flatter divergence angle shown in Figure 504.4. Single lane loop connectors may use a diverge angle of as much as that shown on Figure 504.2B for ramps, if necessary. The choice will depend upon interchange configuration and driver expectancy. Single lane connectors in excess of 1,000 feet in length should be widened to two lanes to provide for passing maneuvers (see Index 504.4(4)).

- (6) *Branch Connections.* A branch connection is defined as a multilane connection between two freeways. A branch connection should be provided when the design year volume exceeds 1500 equivalent passenger cars per hour.

Merging branch connections should be designed as shown in Figure 504.3L. Diverging branch connections should be designed as shown in Figure 504.4. The diverging branch connection leaves the main freeway lanes on a flatter angle shown in Figure 504.4 than the standard 2-lane ramp exit connection shown in Figure 504.3K. The standard ramp exit connects to a local street. The diverging branch connection connects to another freeway and has a flatter angle that allows a higher departure speed.

At a branch merge, a 2,500-foot length of auxiliary lane should be provided beyond the merge of one lane of the inlet, except where it does not appear that capacity on the freeway will be reached until five or more years after the 20 year design period. In this case the length of auxiliary lane should be a minimum of 1,000 feet. For diverging connections where less than capacity conditions beyond the design year are anticipated, the length of auxiliary lane in advance of the exit should be 1,300 feet.

- (7) *Lane Drops.* The lane drop taper on a freeway-to-freeway connector should not be less than WV.
- (8) *Metering.* Any decision to meter freeway-to-freeway connectors must be carefully considered as driver expectancy on these types of facilities is for high-speed uninterrupted flow. If metering is anticipated on a

connector, discussions with the HQ Traffic Liaison and Design Coordinator should take place as early as possible. Issues of particular concern are adequate deceleration lengths to the end of the queue, potential need to widen shoulders if sight distance is restricted (particularly on-ramps with 5-foot shoulders on each side), and the potential for queuing back onto the freeway.

504.5 Auxiliary Lanes

In order to ensure satisfactory operating conditions, auxiliary lanes may be added to the basic width of traveled way.

Where an entrance ramp of one interchange is closely followed by an exit ramp of another interchange, the acceleration and deceleration lanes should be joined with an auxiliary lane. Auxiliary lanes are frequently used when the weaving distance, measured as shown in Figure 504.2A is less than 2,000 feet. Where interchanges are more widely spaced and ramp volumes are high, the need for an auxiliary lane between the interchanges should be determined in accordance with Index 504.7.

Auxiliary lanes may be used for the orientation of traffic at 2-lane ramps or branch connections as illustrated on Figure 504.3L and Figure 504.4. The length and number of auxiliary lanes in advance of 2-lane exits are based on percentages of turning traffic and a weaving analysis.

Auxiliary lanes should be considered on all freeway entrance ramps with significant truck volumes. The grade, volumes and speeds should be analyzed to determine the need for auxiliary lanes. An auxiliary lane would allow entrance ramp traffic to accelerate to a higher speed before merging with mainline traffic, or simply provide more opportunity to merge. See Index 504.2 for specific requirements.

504.6 Mainline Lane Reduction at Interchanges

The basic number of mainline lanes should not be dropped through a local service interchange. The same standard should also be applied to freeway-to-freeway interchanges where less than 35 percent of the traffic is turning (see Figure 504.4). Where

movements operate at least one level of service better than the mainline level of service. In determining acceptable hourly operating volumes, peak hour factors should be used.

The minimum weaving length, measured as shown on Figures 504.2A and 504.2B shall be 2,000 feet in urban areas, 5,000 feet outside urban areas, and 5,000 feet between freeway-to-freeway interchanges and other interchanges. The volumes used must be volumes unconstrained by metering regardless of whether metering will be used. It should be noted that a weaving analysis must be considered over an entire freeway segment as weaving can be affected by other nearby ramps.

The District Traffic Operations Branch should be consulted for difficult weaving analysis problems.

504.8 Access Control

Access rights shall be acquired along interchange ramps to their junction with the nearest public road. At such junctions, for new construction, access control should extend 100 feet beyond the end of the curb return or ramp radius in urban areas and 300 feet in rural areas, or as far as necessary to ensure that entry onto the facility does not impair operational characteristics. Access control shall extend at least 50 feet beyond the end of the curb return, ramp radius, or taper.

Typical examples of access control at interchanges are shown in Figure 504.8. These illustrations do not presume to cover all situations or to indicate the most desirable designs for all cases. When there is state-owned access control on both sides of a local road, a maintenance agreement may be needed.

For new construction or major reconstruction, access rights shall be acquired on the opposite side of the local road from ramp terminals to preclude driveways or local roads within the ramp intersection. This access control would limit the volume of traffic and the number of phases at the intersection of the ramp and local facility, thereby optimizing capacity and operation of the ramp. Through a combination of access control and the use of raised median islands along the local facility, right-in/right-out access may be permitted beyond 200 feet from the ramp intersection. The length of access control on both

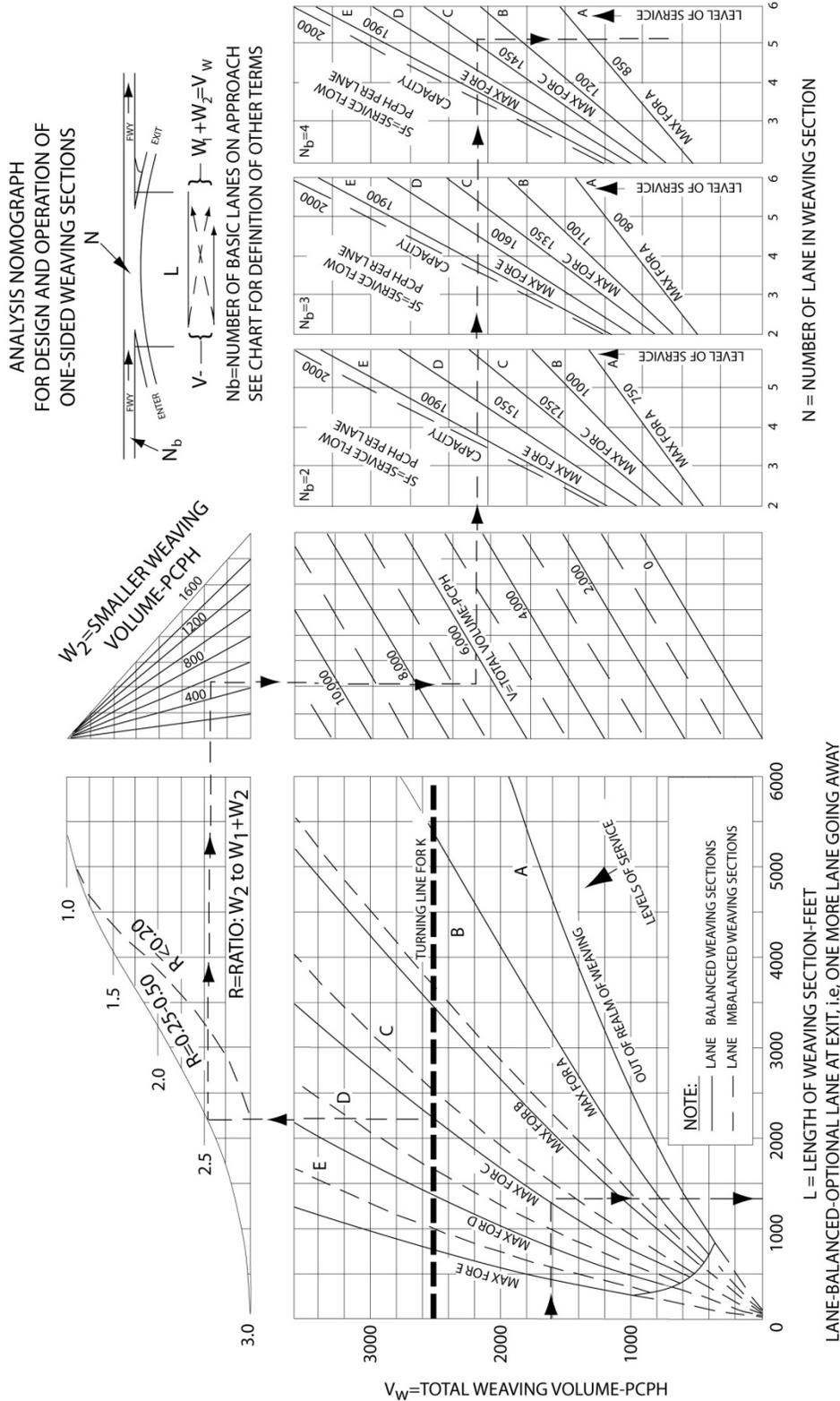
sides of the local facility should match. See 504.3(3) for local road intersection.

In Case 2 consider private ownership within the loop only if access to the property is an adequate distance from the ramp junction to preserve operational integrity.

In Case 3 if the crossroads is near the ramp junction at the local road, full access control should be acquired on the local road from the junction to the intersection with the crossroad.

Case 6 represents a slip ramp design. If the ramp is perpendicular to the local/frontage road refer to Case 3. In Case 6 if the crossroad is near the ramp junction to the local/frontage road, access control should be acquired on the opposite side of the local road from the junction.

Figure 504.7A
Design Curve for Freeway and Collector Weaving



NOTE: EXTRAPOLATION OF CHART BEYOND THE BOUNDARIES GIVEN IS NOT ADVISED.

Example: The nomograph is entered on the left (see dashed line and arrows) with weaving volume, W_1+W_2 , (or V_w) followed by projection to the right, intersecting the desired weaving LOS; a vertical drop from this point provides weaving distance $L = 1300$ ft. Returning to first intersection point of V_w with LOS line, an upward projection along the LOS line is intersected with the horizontal, heavy dashed, "turning line for K' " from here the solution line is extended vertically to intersect the K values curve, from which a horizontal extension meets the desired W_2 volume. Then a downward turn to total volume, V , from which the line is horizontally projected to the right, intersection (in this case) the desired LOS = C curve having an SF of 1450 (representing the overall or composite operation of the weaving section), from which a downward extension yields a N of 5.2; this would be rounded to $N = 5$ lanes.

CHAPTER 810 HYDROLOGY

Topic 811 - General

Index 811.1 - Introduction

Hydrology is often defined as: "A science dealing with the properties, distribution, and circulation of water on the surface of the land, in the soil and underlying rocks, and in the atmosphere." This is a very broad definition encompassing many disciplines relating to water. The highway engineer is principally concerned with surface hydrology and controlling surface runoff. Controlling runoff includes the hydraulic design of drainage features for both cross highway drainage (Chapter 820) and removal of runoff from the roadway (Chapter 830).

The runoff of water over land has long been studied and some rather sophisticated theories and methods have been proposed and developed for estimating flood flows. Most attempts to describe the process have been only partially successful at best. This is due to the complexity of the process and interactive factors. The random nature of rainfall, snowmelt, and other sources of water further complicate the process.

It should be understood that there are no exact methods for hydrologic analysis. Different methods that are commonly used may produce significantly different results for a specific site and particular situation.

Although hydrology is not an exact science, it is possible to obtain solutions which are functionally acceptable to form the basis for design of highway drainage facilities.

More complete information on the principles and engineering techniques pertaining to hydrology for transportation and highway engineers may be found in FHWA Hydraulic Design Series (HDS) No. 2, Highway Hydrology.

This chapter will focus primarily on the hydrologic analyses that are conducted for peak flow facilities for both transportation facility and cross drainage. In many cases, these peak flow facilities serve dual purposes and receive and convey storm water flows while meeting water quality criteria and other flow criteria independent of Chapter 810. Information

related to the designer's responsibility for the hydrologic design of storm water flow facilities is contained in the Department's Project Planning and Design Guide. See: <http://www.dot.ca.gov/hq/oppd/stormwtr/ppdg.htm>

811.2 Objectives of Hydrologic Analysis

Regardless of the size or cost of the drainage feature the most important step prior to hydraulic design is estimating the discharge (rate of runoff) or volume of runoff that the drainage facility will be required to convey or control.

While some hydrologic analysis is necessary in establishing the quantity of surface water that must be considered in the design of all highway drainage facilities, the extent of such studies are to be commensurate with the importance of the highway, the potential for damage to the highway, loss of property, and hazard to life associated with the facilities.

The choice of analytical method must be a conscious decision made as each problem arises. To make an informed decision, the highway engineer must determine:

- What level of hydrologic analysis is justified.
- What data are available or must be collected.
- What methods of analysis are available including the relative strengths and weaknesses in terms of cost and accuracy.

Cross drainage design, Chapter 820, normally requires more extensive hydrologic analysis than is necessary for roadway drainage design, Chapter 830. The well known and relatively simple "Rational Method" (see Index 819.2) is generally adequate for estimating the rate or volume of runoff for the design of on-site roadway drainage facilities and removal of runoff from highway pavements.

811.3 Peak Discharge

Peak discharge is the maximum rate of flow of water passing a given point during or after a rainfall event. Peak discharge, often called peak flow, occurs at the momentary "peak" of the stream's flood hydrograph. (See Index 816.5, Flood Hydrograph.)

Design discharge, expressed as the quantity (Q) of flow in cubic feet per second (CFS), is the peak

discharge that a highway drainage structure is sized to handle. Peak discharge is different for every storm and it is the highway engineer's responsibility to size drainage facilities and structures for the magnitude of the design storm and flood severity. The magnitude of peak discharge varies with the severity of flood events which is based on probability of exceedance (see Index 811.4). The selection of design storm frequency and flood probability are more fully discussed under Topic 818, Flood Probability and Frequency.

811.4 Flood Severity

Flood severity is usually stated in terms of:

- Probability of Exceedance, or
- Frequency of Recurrence.

Modern concepts tend to define a flood in terms of probability. Probability of exceedance, the statistical odds or chance of a flood of given magnitude being exceeded in any year, is generally expressed as a percentage. Frequency of recurrence is expressed in years, on the average, that a flood of given magnitude would be predicted. Refer to Topic 818 for further discussion of flood probability and frequency.

811.5 Factors Affecting Runoff

The highway engineer should become familiar with the many factors or characteristics that affect runoff before making a hydrologic analysis. The effects of many of the factors known to influence surface runoff only exist in empirical form. Extensive field data, empirically determined coefficients, sound judgment, and experience are required for a quantitative analysis of these factors. Relating flood flows to these causative factors has not yet advanced to a level of precise mathematical expression.

Some of the more significant factors which affect the hydraulic character of surface water runoff are categorized and briefly discussed in Topics 812 through 814. It is important to recognize that the factors discussed may exist concurrently within a watershed and their combined effects are very difficult to quantify.

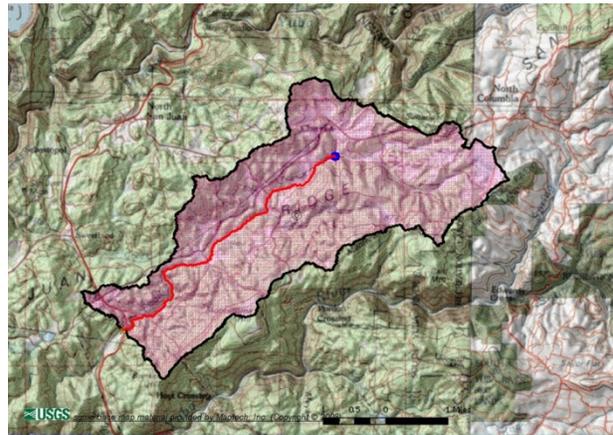
Topic 812 - Basin Characteristics

812.1 Size

The size (area) of a drainage basin is the most important watershed characteristic affecting runoff. Determining the size of the drainage area that contributes to flow at the site of the drainage structure is a basic step in a hydrologic analysis regardless of the method used to evaluate flood flows. The drainage area typically expressed in acres or square miles, is frequently determined from digital elevation maps (DEMs), field surveys, topographic maps, or aerial photographs. Automated watershed delineation is included within several of the software programs indicated under the "Hydrology" column of Table 808.1, e.g., USGS StreamStats and WMS. See Figure 812.1.

Figure 812.1

Automated Watershed Delineation



812.2 Shape

The shape, or outline formed by the basin boundaries, affects the rate at which water is supplied to the main stream as it proceeds along its course from the runoff source to the site of the drainage structure. Long narrow watersheds generally give lower peak discharges than do fan or pear shaped basins.

812.3 Slope

The slope of a drainage basin is one of the major factors affecting the time of overland flow and concentration of rainfall (see Index 816.6, Time of

Concentration). Steep slopes tend to result in shorter response time and increase the discharge while flat slopes tend to result in longer response time and reduce the discharge. Automated basin slope calculation is included within several of the software programs indicated under the "Hydrology" column of Table 808.1, e.g., USGS StreamStats and WMS.

812.4 Land Use

Changes in land use nearly always cause increases in surface water runoff. Of all the land use changes, urbanization is the most dominant factor affecting the hydrology of an area.

Land use studies may be necessary to define present and future conditions with regard to urbanization or other changes expected to take place within the drainage basin.

Valuable information concerning land use trends is available from many sources such as:

- State, regional or municipal planning organizations.
- U.S. Geological Survey.
- U.S. Department of Agriculture Economic Research Service.

Within each District there are various organizations that collect, publish or record land use information. The District Hydraulics Engineer should be familiar with these organizations and the types of information they have available.

A criterion of good drainage design is that future development and land use changes which can reasonably be anticipated to occur during the design life of the drainage facility be considered in the hydraulic analysis and estimation of design discharge.

812.5 Soil and Geology

The type of surface soil which is characteristic of an area is an important consideration for any hydrologic analysis and is a basic input to the National Resources Conservation Service (NRCS) method. Rock formations underlying the surface soil and other geophysical characteristics such as volcanic, glacial, and river deposits can have a significant effect on run-off.

The major source of soil information is the National Resources Conservation Service (NRCS) of the U.S. Department of Agriculture.

Use the following link to access soil information at the NRCS Web Soil Survey website: <http://websoilsurvey.nrcs.usda.gov/app/>.

812.6 Storage

Interception and depression storage are generally not important considerations in highway drainage design and may be ignored in most hydrologic analysis. Interception storage is rainfall intercepted by vegetation and never becomes run-off. Depression storage is rainfall lost in filling small depressions in the ground surface, storage in transit (overland or channel flow), and storage in ponds, lakes or swamps.

Detention storage can have a significant effect in reducing the peak rate of discharge, but this is not always the case. There have been rare instances where artificial storage radically redistributes the discharges and higher peak discharges have resulted than would occur had the storage not been added.

The effect of flood-control reservoirs should be considered in evaluating downstream conditions, flood peaks, and river stages for design of highway structures. The controlling public agency or the owner should be contacted for helpful information on determining the effects, if any, on downstream highway drainage structures.

It is not uncommon for flood control projects to be authorized but never constructed because funds are not appropriated. Therefore a flood control project should exist or be under construction if its effects on a drainage system are to be considered.

812.7 Elevation

The mean elevation of a drainage basin and significant variations in elevation within a drainage basin may be important characteristics affecting run-off, particularly with respect to precipitation falling as snow. Elevation is a basic input to some of the USGS Regional Regression Equations (see Index 819.2(2)).

812.8 Orientation

The amount of runoff can be affected by the orientation of the basin. Where the general slope of

the drainage basin is to the south it will receive more exposure to the heat of the sun than will a slope to the north. Such orientation affects transpiration, evaporation, and infiltration losses. Snowpack and the rate at which snow melts will also be affected. A basin's orientation with respect to the direction of storm movement can affect a flood peak. Storms moving upstream produce lower peaks than storms tending to move in the general direction of stream flow.

Topic 813 - Channel and Floodplain Characteristics

813.1 General

Streams are formed by the gathering together of surface waters into channels that are usually well defined. The natural or altered condition of the channels can materially affect the volume and rate of runoff and is a significant consideration in the hydrological analysis for cross drainage design.

A useful reference relative to issues associated with transverse and longitudinal highway encroachments upon river channels and floodplains is the FHWA Hydraulic Design Series (HDS) No. 6 "River Engineering for Highway Encroachments."

813.2 Length and Slope

The longer the channel the more time it takes for water to flow from the beginning of the channel to the site under consideration. Channel length and effective channel slope are important parameters in determining the response time of a watershed to precipitation events of given frequency.

In the case of a wide floodplain with a meandering main channel the effective channel length will be reduced during flood stages when the banks are overtopped and flow tends more toward a straight line.

813.3 Cross Section

Flood peaks may be estimated by using data from stream gaging stations and natural channel cross section information.

Although channel storage is usually ignored in the hydrologic analysis for the design of highway drainage structures, channel cross section may significantly affect discharge, particularly in wide floodplains with heavy vegetation.

If channel storage is considered to be a significant factor, the assistance of an expert in combining the analysis of basin hydrology and stream hydraulics should be sought. The U.S. Army Corps of Engineers has developed HEC-HMS Flood Hydrograph Package and HEC-RAS, Water Surface Profiles, for this type of analysis. For modeling complex water surface profiles, where one-dimensional models fail, the Finite Element Surface Water Modeling System Two Dimensional Flow in a Horizontal Plane (SMS) was developed by others.

813.4 Hydraulic Roughness

Hydraulic roughness represents the resistance to flows in natural channels and floodplains. It affects both the time response of a drainage channel and channel storage characteristics. The lower the roughness, the higher the peak discharge and the shorter the time of the resulting hydrograph. The total volume of runoff however is virtually independent of hydraulic roughness.

Streamflow is frequently indirectly computed by using Manning's equation, see Index 866.3(4). Procedures for selecting an appropriate coefficient of hydraulic roughness, Manning's "n", may be found in the FHWA report, "Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains". See <http://www.fhwa.dot.gov/bridge/wsp2339.pdf>

813.5 Natural and Man-made Constrictions

Natural constrictions, such as gravel bars, rock outcrops and debris jams as well as artificial constrictions such as diversion and storage dams, grade-control structures, and other water-use facilities may control or regulate flow. Their effect on the flood peak may be an important consideration in the hydrologic analysis.

813.6 Channel Modifications

Channel improvements such as channel-straightening, flood control levees, dredging, bank clearing and removal of obstructions tend to reduce natural attenuation and increase downstream flood peaks.

813.7 Aggradation - Degradation

Aggradation, deposited sediments, may lessen channel capacity and increase flood heights causing

overflow at a lower discharge. Degradation, the lowering of the bed of a stream or channel, may increase channel capacity and result in a higher peak discharge.

The validity of hydrologic analysis using observed historical highwater marks may be affected by aggradation or degradation of the streambed. The effects of aggradation and degradation are considerations in selecting an effective drainage system design to protect highways and adjacent properties from damage. For more information refer to the FHWA report entitled, "Stream Channel Degradation and Aggradation: Causes and Consequences to Highways." See <http://isddc.dot.gov/OLPFiles/FHWA/009471.pdf>.

813.8 Debris

The quantity and size of solid matter carried by a stream may affect the hydrologic analysis of a drainage basin. Bulking due to mud, suspended sediment and other debris transported by storm runoff may significantly increase the volume of flow, affect flow characteristics, and can be a major consideration in the hydraulic design of drainage structures. In particular, bulking factors are typically a consideration in determining design discharges for facilities with watersheds that are located within mountainous regions subject to fire and subsequent soil erosion (see Figure 813.1), or in arid regions when the facility is in the vicinity of alluvial fans (see Index 819.7(2) and Index 872.3(5) for special considerations given to highways located across desert washes).

Debris control methods, structures, and design considerations are discussed in Topic 822, Debris Control.

The District Hydraulics Engineer should be consulted for any local studies that may be available. If both stream gage data and local studies are available, a determination of whether post-fire peak flows are included within the data record should be made. Consideration should be given to treating a significant post-fire peak as the design discharge in lieu of the peak discharge obtained through gage analysis for a given probability flood event. Records of stream discharge from burned and long-unburned (unburned for 40 years or more years) areas have showed peak discharge increases from 2 to 30 times in the first year after burning. In

mountainous regions subject to fire with no local studies available, the U.S. Forest Service should be contacted for fire history in order to determine if there is a significant post-fire peak within the stream records.

Figure 813.1
Post-Fire Debris



Alamos Canyon, Ventura County, post-fire debris and plugged culvert barrels (Highway 118)

Topic 814 - Meteorological Characteristics

814.1 General

Meteorology is the science dealing with the earth's atmosphere, especially the weather. As applied to hydrology for the highway designer the following elements of meteorological phenomena are considered the more important factors affecting runoff and flood predictions.

814.2 Rainfall

Rainfall is the most common factor used to predict design discharge. Unfortunately, due to the many interactive factors involved, the relationship between rainfall and runoff is not all that well defined. Intuitively, engineers know and studies confirm, that runoff increases in proportion to the rainfall on a drainage basin. Highway design engineers are cautioned about assuming that a given frequency storm always produces a flood of the same frequency. There are analytical techniques for ungaged watersheds that are based on this assumption. A statistical analysis of extensive past rainfall records should be made before such a correlation is accepted.

Rainfall event characteristics which are important to highway drainage design are:

- Intensity (rate of rainfall)
- Duration (time rainfall lasts)
- Frequency (statistical probability of how often rainfall will occur)
- Time Distribution (intensity hyetograph)
- Storm Type (orographic, convective or cyclonic)
- Storm Size (localized or broad areal extent)
- Storm Movement (direction of storm)

814.3 Snow

Much of the precipitation that falls in the mountainous areas of the state falls as frozen water in the form of snow, hail, and sleet. Since frozen precipitation cannot become part of the runoff until melting occurs it is stored as snowpack until thawed by warmer weather.

Rain upon an accumulation of snow can cause a much higher peak discharge than would occur from rainfall alone. The parameters of snow which may need to be considered in quantifying peak flood runoff are:

- Mean annual snowfall
- Water content of snowpack
- Snowmelt rate

814.4 Evapo-transpiration

Evaporation and transpiration are two natural processes by which water reaching the earth's surface is returned to the atmosphere as vapor. The losses due to both phenomena are important to long term hydrology and water balance in the watershed and are usually ignored in the hydrologic analysis for the design of highway drainage facilities.

814.5 Tides and Waves

The combined effect of upland runoff and tidal action is a primary consideration in the design of highway drainage structures and shore protection facilities along the coastlines, on estuaries, and in river delta systems.

The time and height of high and low water caused by the gravitational attraction of the sun and moon upon the earth's oceans are precisely predictable. Information on gravitational tides and tidal bench marks for the California Coastline is available from the following report: http://www.slc.ca.gov/reports/ca_marine_boundary_program_final_report.pdf or from the following web-site: <http://co-ops.nos.noaa.gov/sitemap.html>.

One of the most devastating forces affecting the coastline occurs when an astronomical high tide and a storm of hurricane proportion arrive on the land at the same time. This is also true of the effect of a tsunami. A tsunami is a wave caused by an earthquake at sea. If shore protection were designed to withstand the forces of a tsunami, it would be extremely costly to construct. Since it would be so costly and the probability of occurrence is so slight, such a design may not be justified.

Wind-waves directly affect coastal structures and cause dynamic changes in coastal morphology. The U.S. Corps of Engineers collects and publishes data which may be used to predict size of Pacific Coast wind-waves. Information pertaining to the California coastline from the Mexican border north to Cape San Martin can be obtained from:

U.S. Army Corps of Engineers
 Los Angeles District
 915 Wilshire Blvd., Suite 1101
 Los Angeles, CA 90017
 (213) 452-3333

For information from Cape San Martin to the Oregon border from:

U.S. Army Corps of Engineers
San Francisco District
1455 Market Street
San Francisco, CA 94103-1398
(415) 503-6804

Also see the following website for USGS Coastal Storm Modeling System (CoSMoS) for detailed predictions of storm-induced coastal flooding, erosion and cliff failures over large geographic scales:

http://walrus.wr.usgs.gov/coastal_processes/cosmos/

Wind-waves are also generated on large inland bodies of water and their effect should be considered in the design of shoreline highway facilities.

Topic 815 - Hydrologic Data

815.1 General

The purpose for which a hydrologic study is to be made will determine the type and amount of hydrologic data needed. The accuracy necessary for preliminary studies is usually not as critical as the desirable accuracy of a hydrologic analysis to be used for the final design of highway drainage structures. If data needs can be clearly identified, data collection and compilation efforts can be tailored to the importance of the project.

Data needs vary with the methods of hydrologic analysis. Highway engineers should remember that there is no single method applicable to all design problems. They should make use of whatever hydrologic data that has been developed by others whenever it is available and applicable to their needs.

Frequently there is little or no data available in the right form for the project location. For a few locations in the State, so much data has been compiled that it is difficult to manage, store, and retrieve the information that is applicable to the project site.

815.2 Categories

For most highway drainage design purposes there are three primary categories of hydrologic data:

- (1) *Surface Water Runoff.* This includes daily and annual averages, peak discharges, instantaneous values, and highwater marks.
- (2) *Precipitation.* Includes rainfall, snowfall, hail, and sleet.
- (3) *Drainage Basin Characteristics.* Adequate information may not be readily available but can generally be estimated or measured from maps, field reviews or surveys. See Topic 812 for a discussion of basin characteristics.

Other special purpose categories of hydrologic data which may be important to specific problems associated with a highway project are:

- Sediment and debris transport
- Snowpack variations
- Groundwater levels and quantity
- Water quality

815.3 Sources

Hydrologic data necessary for the design of cross drainage (stream crossings) are usually obtained from a combination of sources.

- (1) *Field Investigations.* A great deal of the essential information can only be obtained by visiting the site. Except for extremely simple designs or the most preliminary analysis, a field survey or site investigation should always be made.

To optimize the amount and quality of the hydrologic data collected, the field survey should be well planned and conducted by an engineer with general knowledge of drainage design. Data collected are to be documented. When there is reason to believe that sensitive resources or unusual site conditions may exist, preparation of a written report with maps and photographs may be appropriate. See Topic 804 for Floodplain Encroachments. Index 3.1.1 of HDS No. 2 discusses site investigations and field surveys. Typical data collected in a field survey are:

- Highwater marks
- Performance and condition of existing drainage structures
- Stream alignment

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- Stream stability and scour potential
- Land use and potential development
- Location and nature of physical and cultural features
- Vegetative cover
- Upstream constraints on headwater elevation
- Downstream constraints
- Debris potential

(2) *Federal Agencies.* The following agencies collect and disseminate stream flow data:

- Geological Survey (USGS)
- Corps of Engineers (COE)
- Bureau of Reclamation (USBR)
- National Resources Conservation Service (NRCS)
- Forest Service (USFS)
- Bureau of Land Management (BLM)
- Federal Emergency Management Agency (FEMA)
- Environmental Protection Agency (EPA)

The USGS is the primary federal agency charged with collecting and maintaining water related data. Stream-gaging station data and other water related information collected by the USGS is published in Water Supply Papers and through the USGS Office of Surface Water website. The USGS web-based tool StreamStats provides streamflow statistics, drainage-basin characteristics, and other information for user-selected sites on streams. See <http://water.usgs.gov/osw/streamstats>.

(3) *State Agencies.* The primary state agency collecting stream-gaging and precipitation (rain-gage and snowfall) data is the California Department of Water Resources (DWR). The California Data Exchange Center (CDEC) installs, maintains, and operates an extensive hydrologic data collection network including automatic snow reporting gages and precipitation and river stage sensors. See <http://cdec.water.ca.gov/index.html>.

(4) *Local Agencies.* Entities such as cities, counties, flood control districts, or local improvement districts study local drainage conditions and are often a valuable source of hydrologic data.

(5) *Private Sector.* Water using industries or utilities, railroads and local consultants frequently have pertinent hydrologic records and studies available.

815.4 Stream Flow

Once surface runoff water enters into a stream, it becomes "stream flow". Stream flow is the only portion of the hydrologic cycle in which water is so confined as to make possible reasonably accurate measurements of the discharges or volumes involved.

The two most common types of stream flow data are:

- Gaging Stations - data generally based on recording gage station observations with detailed information about the stream channel cross section. Current meter measurements of transverse channel velocities are made to more accurately reflect stream flow rates.
- Historic - data based on observed high water mark and indirect stream flow measurements.

Stream flow data are usually available as mean daily flow or peak daily flow. Daily flow is a measurement of the rate of flow in cubic feet per second (CFS) for the 24-hour period from midnight to midnight.

"Paleoflood" (ancient flood) data has been found useful in extending stream gaging station records. (See Topic 817 for further discussion on measuring stream flow)

815.5 Precipitation

Precipitation data is collected by recording and non-recording rain gages. Precipitation collected by vertical cylindrical rain gages is designated as "point rainfall".

Regardless of the care and precision used, precipitation measurements from rain gages have inherent and unavoidable shortcomings. Snow and wind problems frequently interrupt rainfall records.

Extreme precipitation data from recording rain gage charts are generally underestimated.

Rain gage measurements are seldom used directly by highway engineers. The statistical analysis which must be done with precipitation measurements is nearly always performed by qualified hydrologists and meteorologists.

NOAA's Atlas 14 is an example of precipitation data that has been converted into formats usable by designers. See <http://hdsc.nws.noaa.gov/hdsc/pfds/>.

815.6 Adequacy of Data

All hydrologic data that has been collected must be evaluated and compiled into a usable format. Experience, knowledge and judgment are an important part of data evaluation. It must be ascertained whether the data contains inconsistencies or other unexplained anomalies which might lead to erroneous calculations and conclusions that could result in the over design or under design of drainage structures.

Topic 816 - Runoff

816.1 General

The process of surface runoff begins when precipitation exceeds the requirements of:

- Vegetal interception.
- Infiltration into the soil.
- Filling surface depressions (puddles, swamps and ponds). As rain continues to fall, surface waters flow down slope toward an established channel or stream.

816.2 Overland Flow

Overland flow is surface waters which travel over the ground as sheet flow, in rivulets and in small channels to a watercourse.

816.3 Subsurface Flow

Waters which move laterally through the upper soil surface to streams are called "interflow" or "subsurface flow". For the purpose of highway drainage hydrology, where peak design discharge (flood peaks) are the primary interest, subsurface flows are considered to be insignificant. Subsurface flows travel slower than overland flow.

While groundwater and subsurface water may be ignored for runoff estimates, their detrimental effect upon highway structural section stability cannot be overstated. See Chapter 840, Subsurface Drainage.

816.4 Detention and Retention

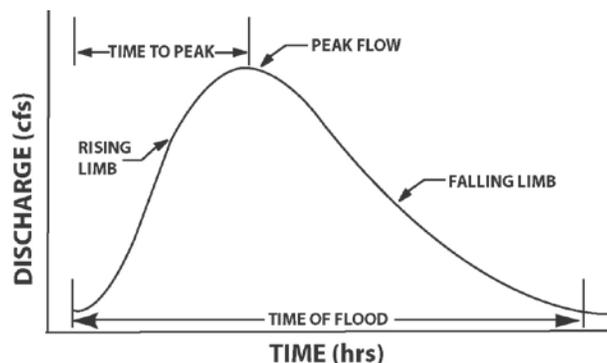
Water which accumulates and ponds in low points or depressions in the soil surface with no possibility for escape as runoff is in retention storage. Where water is moving over the land it is in detention storage. Detained water, as opposed to retained water, contributes to runoff.

816.5 Flood Hydrograph and Flood Volume

In response to a rainstorm the quantity of water flowing in a stream increases. The water level rises and may continue to do so after rainfall ceases. The response of an affected stream, during and after a storm event, can be pictured by plotting discharge against time to produce a flood hydrograph. The principal elements of a typical flood hydrograph are shown in Figure 816.5.

Figure 816.5

Typical Flood Hydrograph



Flood volume is the area under the flood hydrograph. Although flood volume is not considered in the design for all highway drainage facilities, it is an essential design parameter when storage must be evaluated.

Comprehensive guidance on flood hydrographs and methods to estimate the hydrograph may be found in Chapters 6, 7 and 8 of HDS No. 2, Hydrology, and in Chapters 4, 5, 6, 7 and 8 of the user guide for HEC-HMS. See:

[http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_\(CPD-74B\).pdf](http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_(CPD-74B).pdf)

See Index 819.4 for a general discussion of hydrograph methods.

816.6 Time of Concentration (T_c) and Travel Time (T_t)

Time of concentration is defined as the time required for storm runoff to travel from the hydraulically most remote point of the drainage basin to the point of interest.

An assumption made in some of the hydrologic methods for estimating peak discharge, such as the Rational and NRCS Methods (Index 819.2), is that maximum flow results when rainfall of uniform intensity falls over the entire watershed area and the duration of that rainfall is equal to the time of concentration. Time of concentration (T_c) is typically the cumulative sum of three travel times, including:

- Sheet flow
- Shallow concentrated flow
- Channel flow

For all-paved watersheds (e.g., parking lots, roadway travel lanes and shoulders, etc.) it is not necessary to calculate a separate shallow concentrated flow travel time segment. Such flows will typically transition directly from sheet flow to channel flow or be intercepted at inlets with either no, or inconsequential lengths of, shallow concentrated flow.

In many cases a minimum time of concentration will have to be assumed as extremely short travel times will lead to calculated rainfall intensities that are overly conservative for design purposes. For all-paved areas, slopes steeper than 10H:1V, or where there is a limited opportunity for surface storage, a minimum T_c of 5 minutes should be assumed. For rural or undeveloped areas, it is recommended that a minimum T_c of 10 minutes be used for most situations.

Designers should be aware that maximum runoff estimates are not always obtained using rainfall intensities determined by the time of concentration for the total area. Peak runoff estimates may be

obtained by applying higher rainfall intensities from storms of short duration over a portion of the watershed.

(1) *Sheet flow travel time.* Sheet flow is flow of uniform depth over plane surfaces and usually occurs for some distance after rain falls on the ground. The maximum flow depth is usually less than 0.8 inches - 1.2 inches. For unpaved areas, sheet flow normally exists for a distance less than 80 feet - 100 feet. An upper limit of 300 feet is recommended for paved areas.

A common method to estimate the travel time of sheet flow is based on kinematic wave theory and uses the Kinematic Wave Equation:

$$T_t = \frac{0.93L^{3/5} n^{3/5}}{i^{2/5} S^{3/10}}$$

where

- T_t = Travel time in minutes.
- L = Length of flow path in feet.
- S = Slope of flow in feet per foot.
- n = Manning's roughness coefficient for sheet flow (see Table 816.6A).
- i = Design storm rainfall intensity in inches per hour.

If T_t is used (as part of T_c) to determine the intensity of the design storm from the IDF curves, application of the Kinematic Wave Equation becomes an iterative process: an assumed value of T_t is used to determine i from the IDF curve; then the equation is used to calculate a new value of T_t which in turn yields an updated i. The process is repeated until the calculated T_t is the same in two successive iterations.

To eliminate the iterations, use the following simplified form of the Manning's kinematic solution:

$$T_t = \frac{0.42L^{4/5} n^{4/5}}{P_2^{1/2} S^{2/5}}$$

where P₂ is the 2-year, 24-hour rainfall depth in inches (ref. NOAA Atlas 14, <http://hdsc.nws.noaa.gov/hdsc/pfds/>).

The use of flow length alone as a limiting factor for the Kinematic wave equation can lead to circumstances where the underlying assumptions are no longer valid. Over prediction of travel time can occur for conditions with significant amounts of depression storage, where there is a high Manning's *n*-value or for flat slopes. One study suggests that the upper limit of applicability of the Kinematic wave equation is a function of flow length, slope and Manning's roughness coefficient. This study used both field and laboratory data to propose an upper limit of 100 for the composite parameter of $nL/s^{1/2}$. It is recommended that this criteria be used as a check where the designer has uncertainty on the maximum flow length to which the Kinematic wave equation can be applied to project conditions.

Where sheet flow travel distance cannot be determined, a conservative alternative is to assume shallow concentrated flow conditions without an independent sheet flow travel time conditions. See Index 816.6(2).

Table 816.6A
Roughness Coefficients For
Sheet Flow

Surface Description	<i>n</i>
Hot Mix Asphalt	0.011-0.016
Concrete	0.012-0.014
Brick with cement mortar	0.014
Cement rubble	0.024
Fallow (no residue)	0.05
<i>Grass</i>	
Short grass prairie	0.15
Dense grass	0.24
Bermuda Grass	0.41
<i>Woods⁽¹⁾</i>	
Light underbrush	0.40
Dense underbrush	0.80

(1) Woods cover is considered up to a height of 1 inch, which is the maximum depth obstructing sheet flow.

(2) *Shallow concentrated flow travel time.* After short distances, sheet flow tends to concentrate in rills and gullies, or the depth exceeds the

range where use of the Kinematic wave equation applies. At that point the flow becomes defined as shallow concentrated flow. The Upland Method is commonly used when calculating flow velocity for shallow concentrated flow. This method may also be used to calculate the total travel time for both the sheet flow and the shallow concentrated flow segments under certain conditions (e.g., where use of the Kinematic wave equation to predict sheet flow travel time is questionable, or where the designer cannot reasonably identify the point where sheet flow transitions to shallow concentrated flow).

Average velocities for the Upland Method can be taken directly from Figure 816.6 (Source NRCS, National Engineering Handbook part 650) or may be calculated from the following equation:

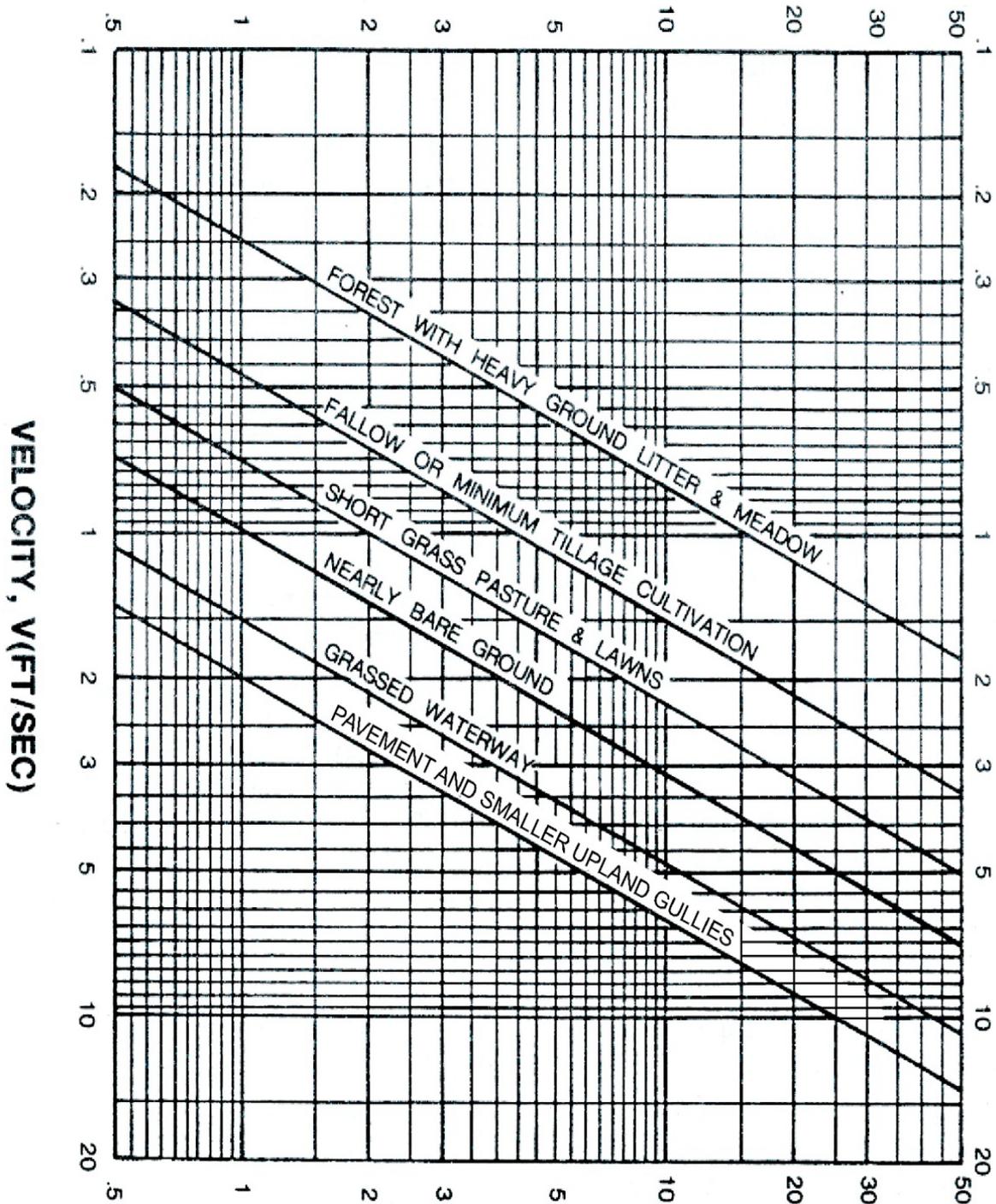
$$V = (3.28) kS^{1/2}$$

Where *S* is the slope in percent and *k* is an intercept coefficient depending on land cover as shown in Table 816.6B. It is assumed that the depth range is 0.1 to 0.2 feet, except for grassed waterways, where the depth range is 0.1 to 0.4 feet,

Table 816.6B
Intercept Coefficients for Shallow
Concentrated Flow

Land cover/Flow regime	<i>k</i>
Forest with heavy ground litter; hay meadow	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland	0.152
Short grass pasture	0.213
Cultivated straight row	0.274
Nearly bare and untilled alluvial fans	0.305
Grassed waterway	0.457
Pavement and small upland gullies	0.620

Figure 816.6
Velocities for Upland Method of
Estimating Travel Time for Shallow Concentrated Flow
WATERCOURSE SLOPE IN PERCENT



The travel time can be calculated from:

$$T_t = \frac{L}{60 V}$$

where T_t is the travel time in minutes, L the length in feet, and V the flow velocity in feet per second.

- (3) *Channel flow travel time.* When the channel characteristics and geometry are known the preferred method of estimating channel flow time is to divide the channel length by the channel velocity obtained by using the Manning's equation, assuming bankfull conditions. See Index 866.3(4), for further discussion of Manning's equation.

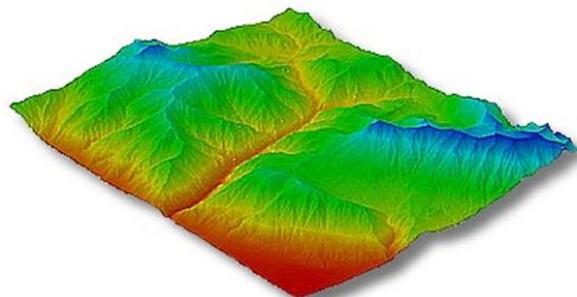
Appropriate values for "n", the coefficient of roughness in the Manning's equation, may be found in most hydrology or hydraulics texts and reference books. Table 866.3A gives some "n" values for lined and unlined channels, gutters, and medians. Procedures for selecting an appropriate hydraulic roughness coefficient may be found in the FHWA report, "Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains." See <http://www.fhwa.dot.gov/bridge/wsp2339.pdf>. Generally, the channel roughness factor will be much lower than the values for overland flow with similar surface appearance.

- (4) *Culvert or Storm Drain Flow.* Flow velocities in a short culvert are generally higher than they would be in the same length of natural channel and comparable to those in a lined channel. In most cases, including short runs of culvert in the channel, flow time calculation will not materially affect the overall time of concentration (T_c). When it is appropriate to separate flow time calculations, such as for urban storm drains, Manning's equation may be used to obtain flow velocities within pipes.

The TR-55 library of equations for sheet flow, shallow concentrated flow and open channel flow is incorporated into the Watershed Modeling System (WMS) for Time of Concentration Calculations using Triangulated Irregular Networks (TINs) and Digital Elevation Maps (DEMs). See Figure 816.7.

Figure 816.7

Digital Elevation Map (DEM)



Topic 817 - Flood Magnitude

817.1 General

The determination of flood magnitude from either measurements made during a flood or after peak flow has subsided requires knowledge of open-channel hydraulics and flood water behavior. There are USGS Publications and other technical references available which outline the procedures for measuring flood flow. However, it is only through experience that accurate measurements can be obtained and/or correctly interpreted.

817.2 Measurements

- (1) *Direct.* Direct flood flow measurements are those made during flood stage. The area and average velocity can be approximated and the estimated discharge can be calculated, from measurements of flow depth and velocity made simultaneously at a number of points in a cross section.

Discharges calculated from continuous records of stage gaging stations are the primary basis for estimating the recurrence interval or frequency of floods. See Figure 817.2.

Figure 817.2 Gaging Station



Smith River Stage Gaging Station at Dr. Fine Bridge

(2) *Indirect.* Indirect flood flow measurements are those made after the flood subsides. From channel geometry measurements and high water marks the magnitude of a flood can be calculated using basic open channel hydraulic equations given in Chapter 860. This method of determining flood discharges for given events is a valuable tool to the highway engineer possessing a thorough knowledge and understanding of the techniques involved. See Figure 817.3.

Figure 817.3 High Water Marks



Topic 818 - Flood Probability And Frequency

818.1 General

The estimation of peak discharges of various recurrence intervals is the most common and important problem encountered in highway engineering hydrology. Since the hydrology for the sizing of highway drainage facilities is concerned with future events, the time and magnitude of which cannot be precisely forecast, the highway engineer must resort to probability statistics to define the design discharge.

Modern hydrologists tend to define floods in terms of probability, as expressed in percentage rather than in terms of return period (recurrence interval). Return period, the "N-year flood", and probability (p) are reciprocals, that is, $p = 1/N$. Therefore, a flood having a 50-year return frequency (Q_{50}) is now commonly expressed as a flood with the probability of recurrence of 0.02 (2 percent chance of being exceeded) in any given year.

There are certain other terminologies which are frequently used and understood by highway engineers but which might have a slight variation in meaning to other engineering branches. For convenience and example, the following definition of terms have been excerpted from Topic 806, Definition of Drainage Terms.

- (1) *Base Flood.* "The flood, tide, or a combination of the two having a 1 percent chance of being exceeded in any given year". The "base flood" is used as the standard flood by FEMA and has been adopted by many agencies for flood hazard analysis to comply with regulatory requirements. See Topic 804, Floodplain Encroachments.
- (2) *Overtopping Flood.* "The flood described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief". The "overtopping flood" is of particular interest to highway drainage engineers because it may be the threshold where the relatively low profile of the highway acts as a flood relief mechanism for the purpose of minimizing upstream

backwater damages. See Figure 818.1. On Interstate highways, CFR 650 states "The design flood for encroachments by through lanes of Interstate highways shall not be less than the flood with a 2-percent chance of being exceeded in any given year. No minimum design flood is specified for Interstate highway ramps and frontage roads or for other highways."

Figure 818.1
Overtopping Flood



(3) *Design Flood.* "The peak discharge (when appropriate, the volume, stage, or wave crest elevation) of the flood associated with the probability of exceedance selected for the design of a highway encroachment". Except for the rare situation where the risks associated with a low water crossing are acceptable, the highway will not be inundated by the "design flood".

(4) *Maximum Historical Flood.* "The maximum flood that has been recorded or experienced at any particular highway location". This information is very desirable and where available is an indication that the flood of this magnitude may be repeated at the project site. Hydrologic analysis may suggest that the probability for recurrence of the "maximum historical flood" is very small, less than 1 percent. Nevertheless consideration should be given to sizing drainage structures to convey the "maximum historical flood". See Figure 818.2.

Figure 818.2
Maximum Historic Flood



(5) *Probable Maximum Flood.* "The flood discharge that may be expected from the most severe combination of critical meteorological and hydrological conditions that are reasonably possible in the region". The "probable maximum flood" is generally not applicable to highway projects. The possibility of a flood of such rare magnitude, as used by the Corps of Engineers, is applicable to projects such as major dams, when consideration is to be given to virtually complete security from potential floods.

818.2 Establishing Design Flood Frequency

There are two recognized alternatives to establishing an appropriate highway drainage design frequency. That is, by policy or by economic analysis. Both alternatives have merit and may be applied exclusively or jointly depending upon general conditions or specific constraints.

Application of traditional predetermined design flood frequencies implies that an acceptable level of risk was considered in establishing the design standard. Modern design concepts, on the other hand, recommend that a range of peak flows be considered and that the design flood be established which best satisfies the specific site conditions and associated risks. A preliminary evaluation of the inherent flood-related risks to upstream and downstream properties, the highway facility, and to the traveling public should be made. This evaluation will indicate whether a predetermined

design flood frequency is applicable or additional study is warranted.

Highway classification is one of the most important factors, but not the sole factor, in establishing an appropriate design flood frequency. Due consideration should be given to all the other factors listed under Index 801.5. If the analysis is correct, the highway drainage system will occasionally be overtaxed. The alternative of accommodating the worst possible event that could happen is usually so costly that it may not be justified.

Highway engineers should understand that the option to select a predetermined design flood frequency is generally only applicable to new highway locations. Because of existing constraints, the freedom to select a prescribed design flood frequency may not exist for projects involving replacement of existing facilities. Caltrans policy relative to up-grading of existing drainage facilities may be found in Index 803.3.

Although the procedures and methodology presented in HEC 17, Design of Encroachments on Flood Plains Using Risk Analysis, are not fully endorsed by Caltrans, the circular is an available source of information on the theory of "least total expected cost (LTEC) design". Highway engineers are cautioned about applying LTEC methodology and procedures to ordinary drainage design problems. The Headquarters Hydraulics Engineer in the Division of Design should be consulted before committing to design by the LTEC method since its use can only be justified and recommended under extra-ordinary circumstances.

818.3 Stationarity and Climate Variability

In Index 818.1, the assumption behind flood probability and frequency analysis is that climate is stationary. Stationarity assumes that hydrology varies within an unchanging envelope of natural variability, so that the past accurately represents the future. It has been a basic assumption used for many years in the planning and design of bridges and culverts and continues to represent the current state of practice that serves the engineering community well.

Climate change as well as better understanding of climate variability have presented a challenge to the validity of this assumption.

Today, there is growing recognition that, despite its successful application in the past, the assumption of stationarity may not accurately represent the future. However, until a multi-disciplinary consensus is reached on future trends that can be expected, stationarity will continue to be utilized with current procedures.

To minimize uncertainty, designers should continue to utilize existing hydrologic tools with the most current datasets available for rainfall and runoff. Observed trends can then be quantified and placed in the context of the uncertainty associated with the frequency estimates themselves.

(1) *Nonstationarity and Climate Variability.* Changes in land use, changing groundwater levels, and urbanization are examples of nonstationarity within a watershed that can affect hydrologic response. The Intergovernmental Panel on Climate Change (IPCC) has stated that "Climate change challenges the traditional assumption that past hydrological experience provides a good guide to future conditions". Although the assumption of stationarity is being challenged, there is no consensus within the scientific or engineering community on a viable replacement.

Topic 819 - Estimating Design Discharge

819.1 Introduction

Before highway drainage facilities can be hydraulically designed, the quantity of run-off (design Q) that they may reasonably be expected to convey must be established. The estimation of peak discharge for various recurrence intervals is therefore the most important, and often the most difficult, task facing the highway engineer. Refer to Table 819.5A for a summary of methods for estimating design discharge.

In Topic 819, various design recommendations are given for both general and region-specific areas of California.

819.2 Empirical Methods

Because the movement of water is so complex, numerous empirical methods have been used in hydrology. Empirical methods in hydrology have great usefulness to the highway engineer. When

correctly applied by engineers knowledgeable in the method being used and its idiosyncrasies, peak discharge estimates can be obtained which are functionally acceptable for the design of highway drainage structures and other features. Some of the more commonly used empirical methods for estimating runoff are as follows.

(1) *Rational Methods.* Undoubtedly, the most popular and most often misused empirical hydrology method is the Rational Formula:

$$Q = CiA$$

Q = Design discharge in cubic feet per second.

C = Coefficient of runoff.

i = Average rainfall intensity in inches per hour for the selected frequency and for a duration equal to the time of concentration. See <http://hdsc.nws.noaa.gov/hdsc/pfds/>

A = Drainage area in acres.

Rational methods are simple to use, and it is this simplicity that has made them so popular among highway drainage design engineers. Design discharge, as computed by these methods, has the same probability of occurrence (design frequency) as the frequency of the rainfall used. Refer to Topic 818 for further information on flood probability and frequency of recurrence.

An assumption that limits applicability is that the rainfall is of equal intensity over the entire watershed. Because of this, Rational Methods should be used only for estimating runoff from small simple watershed areas, preferably no larger than 320 acres. Even where the watershed area is relatively small but complicated by a mainstream fed by one or more significant tributaries, Rational Methods should be applied separately to each tributary stream and the tributary flows then routed down the main channel. Flow routing can best be accomplished through the use of hydrographs discussed in Index 819.4. Since Rational Methods give results that are in terms of instantaneous peak discharge and provide little information relative to runoff rate with respect to time, synthetic hydrographs should be developed for routing significant tributary

inflows. Several relatively simple methods have been established for developing hydrographs, such as transposing a hydrograph from another hydrologically homogeneous watershed. The stream hydraulic method, and upland method are described in HDS No. 2. These, and other methods, are adequate for use with Rational Methods for estimating peak discharge and will provide results that are acceptable to form the basis for design of highway drainage facilities.

It is clearly evident upon examination of the assumptions and parameters which form the basis of the equation that much care and judgment must be applied with the use of Rational Methods to obtain reasonable results.

- The runoff coefficient "C" in the equation represents the percent of water which will run off the ground surface during the storm. The remaining amount of precipitation is lost to infiltration, transpiration, evaporation and depression storage. "C" is a volumetric coefficient that relates the peak discharge to the "theoretical peak" or 100 percent runoff, occurring when runoff matches the net rain rate. Hence "C" is also a function of infiltration and other hydrologic abstractions.

Values of "C" may be determined for un-developed areas from Figure 819.2A by considering the four characteristics of: relief, soil infiltration, vegetal cover, and surface storage.

The designer must use judgment to select the appropriate "C" value within the range. Generally, larger areas with permeable soils, flat slopes and dense vegetation should have the lowest "C" values. Smaller areas with dense soils, moderate to steep slopes, and sparse vegetation should be assigned the highest "C" values.

Some typical values of "C" for developed areas are given in Table 819.2B. Should the basin contain varying amounts of different cover, a weighted runoff coefficient for the entire basin can be determined as:

$$C = \frac{C_1A_1 + C_2A_2 + \dots}{A_1 + A_2 + \dots}$$

- To properly satisfy the assumption that the entire drainage area contributes to the flow;

the rainfall intensity, (i) in the equation expressed in inches per hour, requires that the storm duration and the time of concentration (t_c) be equal. Therefore, the first step in estimating (i) is to estimate (t_c). Methods for determining time of concentration are discussed under Index 816.6.

- Once the time of concentration, (t_c), is estimated, the rainfall intensity, (i), corresponding to a storm of equal duration, may be obtained from available sources such as intensity-duration-frequency (IDF) curves. For IDF curve generating software, see <http://hdsc.nws.noaa.gov/hdsc/pfds/>.

The runoff coefficients given in Figure 819.2A and Table 819.2B are applicable for storms of up to 5 or 10 year frequencies. Less frequent, higher intensity storms usually require modification of the coefficient because infiltration, detention, and other losses have a proportionally smaller effect on the total runoff volume. The adjustment of the rational method for use with major storms can be made by multiplying the coefficient by a frequency factor, $C(f)$. Values of $C(f)$ are given below. Under no circumstances should the product of $C(f)$ times C exceed 1.0.

Frequency (yrs)	$C(f)$
25	1.1
50	1.2
100	1.25

- (2) *Regional Analysis Methods.* Regional analysis methods utilize records for streams or drainage areas in the vicinity of the stream under consideration which would have similar characteristics to develop peak discharge estimates. These methods provide techniques for estimating annual peak stream discharge at any site, gaged or ungaged, for probability of recurrence from 50 percent (2 years) to 1 percent (100 years). Application of these methods is convenient, but the procedure is subject to some limitations.

Regional Flood - Frequency equations developed by the U.S. Geological Survey for use in California are given in Figure 819.2C and

Table 819.7A. These equations are based on regional regression analysis of data from stream gauging stations. The equations in Figure 819.2C were derived from data gathered and analyzed through 2006, while the regions covered by Table 819.7A are reflective of a 1994 study of the Southwestern U.S, which has been supplemented by a more recent 2007 Study of California Desert Region Hydrology. Information on use and development of this method may be found in "Methods for Determining Magnitude and Frequency of Floods in California Based on Data through Water Year 2006" by the U.S. Department of the Interior, Geological Survey.

The Regional Flood-Frequency equations are applicable only to sites within the flood-frequency regions for which they were derived and on streams with virtually natural flows. The equations are not directly applicable to streams in urban areas affected substantially by urban development. In urban areas the equations may be used to estimate peak discharge values under natural conditions and then by use of the techniques described in the publication or HDS No. 2, adjust the discharge values to compensate for urbanization. A method for directly estimating design discharges for some gaged and ungaged streams is also provided in HDS No. 2. The method is applicable to streams on or nearby those for which study data are available.

(3) *Flood Frequency Analysis*

- If there are two gaged sites with similar watershed characteristics but one has a short record and the other has a longer record of peak flows, a two-station comparison analysis can be conducted to extend the equivalent length of record at the shorter gaged site.
- Flood-frequency relations at sites near gaged sites on the same stream (or in a similar watershed) can be estimated using a ratio of drainage area for the ungaged and gaged sites.
- At a gaged site, weighted estimates of peak discharges based on the station flood-frequency relation and the regional

Table 819.2B
Runoff Coefficients for
Developed Areas ⁽¹⁾

Type of Drainage Area	Runoff Coefficient
Business:	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential:	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries:	0.10 - 0.25
Playgrounds:	0.20 - 0.40
Railroad yard areas:	0.20 - 0.40
Unimproved areas:	0.10 - 0.30
Lawns:	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average, 2-7%	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2-7%	0.18 - 0.22
Heavy soil, steep, 7%	0.25 - 0.35
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Drives and walks	0.75 - 0.85
Roofs:	0.75 - 0.95

NOTES:

(1) From HDS No. 2.

Region	Drainage Area (A) mi ²	Mean Annual Precip (P) in	Altitude Index (H) 1000 ft
⁽¹⁾ North Coast	0.2-3000	19-104	0.2-5.7
⁽²⁾ Northeast	0.2-25	all	all
Sierra	0.2-9000	7-85	0.1-9.7
Central Coast	0.2-4000	8-52	0.1-2.4
South Coast	0.2-600	7-40	all
⁽³⁾ South Lahontan- Colorado Desert	N/A	N/A	N/A

Notes:

- (1) In the North Coast region use a minimum value of 1 for altitude index (H)
- (2) See Index 819.7 for hydrologic procedures for those portions of the Northeast Region classified as desert.
- (3) USGS equations not recommended. See Index 819.7

regression equations are considered the best estimates of flood frequency and are used to reduce the time-sampling error that may occur in a station flood-frequency estimate.

- (d) The flood-frequency flows and the maximum peak discharges at several stations in a region should be used whenever possible for comparison with the peak discharge estimated at an ungaged site using a rainfall-runoff approach or regional regression equation. The watershed characteristics at the ungaged and gaged sites should be similar.
- (4) *National Resources Conservation Service (NRCS) Methods.* The Soil Conservation Service's SCS (former title) National Engineering Handbook, 1972, and their 1975, "Urban Hydrology for Small Watersheds", Technical Release 55 (TR-55), present a graphical method for estimating peak discharge. Most NRCS equations and curves provide results in terms of inches of runoff for unit hydrograph development and are not

applicable to the estimation of a peak design discharge unless the design hydrograph is first developed in accordance with prescribed NRCS procedures. NRCS methods and procedures are applicable to drainage areas less than 3 square miles (approx. 2,000 acres) and result in a design hydrograph and design discharge that are functionally acceptable to form the basis for the design of highway drainage facilities.

819.3 Statistical Methods

Statistical methods of predicting stream discharge utilize numerical data to describe the process. Statistical methods, in general, do not require as much subjective judgment to apply as the previously described deterministic methods. They are usually well documented mathematical procedures which are applied to measured or observed data. The accuracy of statistical methods can also be measured quantitatively. However, to assure that statistical method results are valid, the method and procedures used should be verified by an experienced engineer with a thorough knowledge of engineering statistics.

Analysis of gaged data permits an estimate of the peak discharge in terms of its probability or frequency of recurrence at a given site. This is done by statistical methods provided sufficient data are available at the site to permit a meaningful statistical analysis to be made. Water Resources Council Bulletin 17B, 1981, suggests at least 10 years of record are necessary to warrant a statistical analysis. The techniques of inferential statistics, the branch of statistics dealing with the inference of population characteristics, are described in HDS No. 2.

Before data on the specific characteristics to be examined can be properly analyzed, it must be arranged in a systematic manner. Several computer programs are available which may be used to systematically arrange data and perform the statistical computations.

Some common types of data groupings are as follows:

- Magnitude
- Time of Occurrence
- Geographic Location

Several standard frequency distributions have been studied extensively in the statistical analysis of hydrologic data. Those which have been found to be most useful are:

(1) *Log-Pearson Type III Distribution.* The popularity of the Log-Pearson III distribution is simply based on the fact that it very often fits the available data quite well, and it is flexible enough to be used with a wide variety of distributions. Because of this flexibility, the U.S. Water Resources Council recommends its use by all U.S. Government agencies as the standard distribution for flood frequency studies.

The three parameters necessary to describe the Log-Pearson III distribution are:

- Mean flow
- Standard deviation
- Coefficient of skew

Log-Pearson III distributions are usually plotted on log-normal probability graph paper for convenience even though the plotted frequency distribution may not be a straight line.

It should be noted Log-Pearson III analysis is not typically appropriate for desert regions where flood-frequency analysis is complicated due to short annual peak-flow records (usually less than 20 years) and numerous zero flows and (or) low outliers for many stream gages.

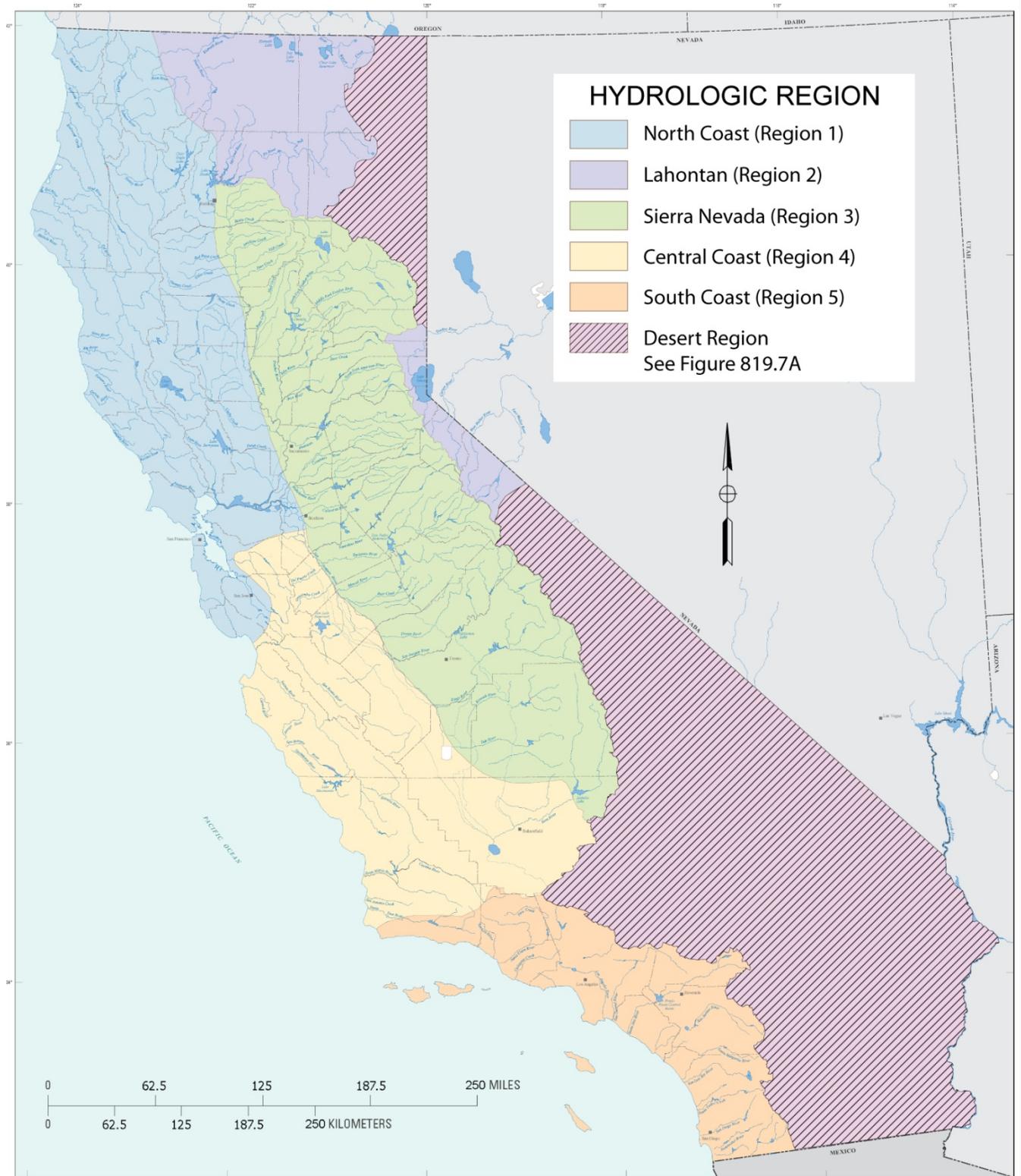
(2) *Log-normal Distribution.* The characteristics of the log-normal distribution are the same as those of the classical normal or Gaussian mathematical distribution except that the flood flow at a specified frequency is replaced with its logarithm and has a positive skew. Positive skew means that the distribution is skewed toward the high flows or extreme values.

(3) *Gumbel Extreme Value Distribution.* The characteristics of the Gumbel extreme value distribution (also known as the double exponential distribution of extreme values) are that the mean flood occurs at the return period of $T_r = 2.33$ years and that it has a positive skew.

Table 819.2C
Regional Flood-Frequency Equations

NORTH COAST (REGION 1)	LAHONTAN (REGION 2)	SIERRA NEVADA (REGION 3)
$Q_2 = 1.82A^{0.904}P^{0.983}$	$Q_2 = 0.0865A^{0.736}P^{1.59}$	$Q_2 = 2.43A^{0.924}E^{-0.646}P^{2.06}$
$Q_5 = 8.11A^{0.887}P^{0.772}$	$Q_5 = 0.182A^{0.733}P^{1.58}$	$Q_5 = 11.6A^{0.907}E^{-0.566}P^{1.70}$
$Q_{10} = 14.8A^{0.880}P^{0.696}$	$Q_{10} = 0.260A^{0.734}P^{1.59}$	$Q_{10} = 17.2A^{0.896}E^{-0.486}P^{1.54}$
$Q_{25} = 26.0A^{0.874}P^{0.628}$	$Q_{25} = 0.394A^{0.733}P^{1.58}$	$Q_{25} = 20.7A^{0.885}E^{-0.386}P^{1.39}$
$Q_{50} = 36.3A^{0.870}P^{0.589}$	$Q_{50} = 0.532A^{0.733}P^{1.58}$	$Q_{50} = 21.1A^{0.879}E^{-0.316}P^{1.31}$
$Q_{100} = 48.5A^{0.866}P^{0.556}$	$Q_{100} = 0.713A^{0.731}P^{1.56}$	$Q_{100} = 20.6A^{0.874}E^{-0.250}P^{1.24}$
CENTRAL COAST (REGION 4)	SOUTH COAST (REGION 5)	<p>Q = Peak discharge in CFS, subscript indicates recurrence interval, in years</p> <p>A = Drainage area, in square miles</p> <p>P = Mean annual precipitation, in inches (Use link to Table 2) http://pubs.usgs.gov/sir/2012/5113/</p> <p>E = Mean basin elevation, in feet</p>
$Q_2 = 0.00459A^{0.856}P^{2.58}$	$Q_2 = 3.60A^{0.672}P^{0.753}$	
$Q_5 = 0.0984A^{0.852}P^{1.97}$	$Q_5 = 7.43A^{0.739}P^{0.872}$	
$Q_{10} = 0.460A^{0.846}P^{1.66}$	$Q_{10} = 6.56A^{0.783}P^{1.07}$	
$Q_{25} = 2.13A^{0.842}P^{1.34}$	$Q_{25} = 4.71A^{0.832}P^{1.32}$	
$Q_{50} = 5.32A^{0.840}P^{1.15}$	$Q_{50} = 3.84A^{0.864}P^{1.47}$	
$Q_{100} = 11.0A^{0.840}P^{0.994}$	$Q_{100} = 3.28A^{0.891}P^{1.59}$	

Figure 819.2C
Regional Flood-Frequency Regions



Special probability paper has been developed for plotting log-normal and Gumbel distributions so that sample data, if it is distributed according to prescribed equations, will plot as a straight line.

- (4) *L-Moments*. L-moments provide an alternative way of describing frequency distributions to traditional product moments (conventional moments) or maximum likelihood approach. They are less susceptible to the presence of outliers in the data than conventional moments and are well suited for the analysis of data that exhibit significant skewness. See overview of methodology used for NOAA Atlas 14 (Index 4.6.1); http://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume6.pdf

819.4 Hydrograph Methods

Hydrograph methods of estimating design discharge relate runoff rates to time in response to a design storm. When storage must be considered, such as in reservoirs, natural lakes, and detention basins used for drainage or sediment control, the volume of runoff must be known. Since the hydrograph is a plot of flow rate against time, the area under the hydrograph represents volume. If streamflow and precipitation records are available for a particular design site, the development of the design hydrograph is a straight forward procedure. Rainfall records can be readily analyzed to estimate unit durations and the intensity which produces peak flows near the desired design discharge.

It often becomes necessary to develop a hydrograph when watersheds have complex runoff characteristics, such as in urban and desert areas or when storage must be evaluated.

Hydrograph methods apply for watersheds in which the time of concentration is longer than the duration of peak rainfall intensity of the design storm. Precipitation applied to the watershed model is uniform spatially, but varies with time. The hydrograph method accounts for losses (e.g., soil infiltration) and transforms the remaining (excess) rainfall into a runoff hydrograph at the outlet of the watershed. There is no size limitation for watershed area. See HDS No. 2; Figure 2-13, for the relationship of discharge and area and effects of basin characteristics on the flood hydrograph.

Hydrographs are also useful for determining the combined rates of flow for two drainage areas which peak at different times. Hydrographs can also be compounded and lagged to account for complex storms of different duration and varying intensities.

See Index 819.7(1)(d) for a detailed discussion on rainfall-runoff simulation for California's Desert regions. The same four general concepts are applicable elsewhere. Other considerations may include:

- Development of a rainfall hyetograph
- Base flow separation
- Direct runoff hydrograph derivation
- Unit hydrograph derivation; and
- Other synthetic unit hydrographs (e.g., Snyder's or Clark's methods)

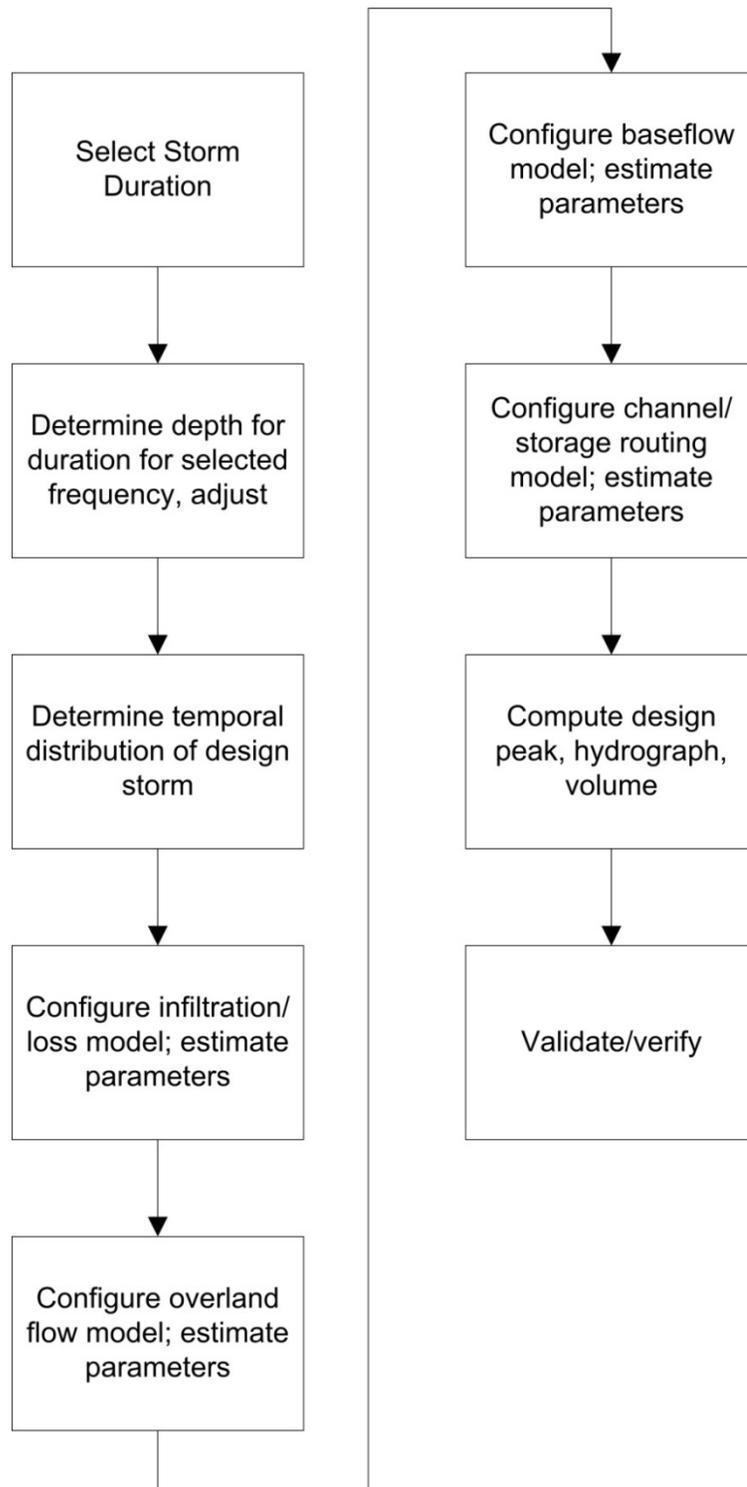
Successful application of most hydrograph methods requires the designer to:

- Define the temporal and spatial distribution of the desired design storm.
- Specify appropriate losses within the model to compute the amount of precipitation lost to other processes, such as infiltration that does not run off the watershed.
- Specify appropriate parameters to compute runoff hydrograph resulting from excess (not lost) precipitation.
- If necessary for the application, specify appropriate parameters to compute the lagged and attenuated hydrograph at downstream locations. Basic steps to developing and applying a rainfall-runoff model for predicting the required design flow are illustrated in Figure 819.4A.

Several methods of developing hydrographs are described in HDS No. 2. For basins without data, two of the most widely used methods described in HDS No. 2 for developing synthetic hydrographs are:

- Unit Hydrograph (UH)
- SCS Triangular Hydrograph

Figure 819.4A

Basic Steps to Developing and Applying a Rainfall-runoff Model for Predicting the Required Design Flow

Both methods however tend to be somewhat inflexible since storm duration is determined by empirical relations.

For basins with data, HEC-HMS includes the following direct runoff models:

- User specified UH
- Parametric and Synthetic UH
- Snyder's UH
- Clark's UH
- ModClark Model
- Kinematic-wave Model

For more information see; Chapters 4, 5, 6, 7 and 8 of the user guide for HEC-HMS. See: [http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_\(CPD-74B\).pdf](http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_(CPD-74B).pdf)

819.5 Transfer of Data

Often the highway engineer is confronted with the problem where stream flow and rainfall data are not available for a particular site but may exist at points upstream or in an adjacent or nearby watersheds.

- (a) If the site is on the same stream and near a gaging station, peak discharges at the gaging station can be adjusted to the site by drainage area ratio and application of some appropriate power to each drainage area. The USGS may be helpful in suggesting appropriate powers to be used for a specific hydrologic region.
- (b) If a design hydrograph can be developed at an upstream point in the same watershed, the procedure described in HDS No. 2 can be used to route the design hydrograph to the point of interest.
- (c) IDF curve generating software, such as NOAA's Atlas 14, have internal routines that provide interstation interpolation that accounts not only for distance from gauge stations, but other factors, such as elevation. No additional effort is required by the designer to address distance/location effects.

819.6 Hydrologic Software

Most simulation models require a significant amount of input data that must be carefully examined by a competent and experienced user with an understanding of the mathematical nuances of the model and the hydrologic nuances of the particular catchment to assure reliable results.

See Table 808.1 for hydrologic software packages that have been reviewed and deemed compatible with Departmental procedures.

A summary of hydrologic software is listed in Table 808.1. Several of those listed are described below.

Watershed Modeling System (WMS) is a comprehensive environment for hydrologic analysis. It was developed by the Engineering Computer Graphics Laboratory of Brigham Young University in cooperation with the U.S. Army Corps of Engineers Waterways Experiment Station (WES).

WMS merges information obtained from terrain models and GIS with industry standard hydrologic analysis models such as HEC-HMS and TR-55.

Terrain models can obtain geometric attributes such as area, slope and runoff distances. Many display options are provided to aid in modeling and understanding the drainage characteristics of terrain surfaces.

WMS uses three primary data sources for model development:

- (1) Geographic Information Systems (GIS) Data
- (2) Digital Elevation Models (DEMs) published by the U.S. Geological Survey (USGS) at both 1:24,000 and 1:250,000 for the entire U.S. (the 1:24,000 data coverage is not complete)
- (3) Triangulated Irregular Networks (TINs)

Automated basin delineation, slope calculation, and basin characteristics are some of the many features available within USGS StreamStats. See; <http://water.usgs.gov/osw/streamstats/>.

AutoDesk Civil 3D/Hydraflow uses NRCS, Rational and Modified Rational methods to generate runoff hydrographs, however, HEC-HMS provides more comprehensive modeling options for runoff and channel flow.

Table 819.5A
Summary of Methods for Estimating Design Discharge

METHOD	ASSUMPTIONS	DATA NEEDS
Rational	<ul style="list-style-type: none"> • Small catchment (< 320 acres) • Concentration time < 1 hour • Storm duration \geq concentration time • Rainfall uniformly distributed in time and space • Runoff is primarily overland flow • Negligible channel storage 	Time of Concentration Drainage area Runoff coefficient Rainfall intensity (http://hdsc.nws.noaa.gov/hdsc/pfds/)
USGS Regional Regression Equations: USGS Water-Resources Investigation 77-21* Improved Highway Design Methods for Desert Storms	<ul style="list-style-type: none"> • Catchment area limit varies by region • Basin not located on floor of Sacramento or San Joaquin Valleys • Peak discharge value for flow under natural conditions unaffected by urban development and little or no regulation by lakes or reservoirs • Ungaged channel 	Drainage area Mean annual precipitation Altitude index
NRCS (TR55)	<ul style="list-style-type: none"> • Small or midsize catchment (< 3 square miles) • Concentration time range from 0.1-10 hour (tabular hydrograph method limit < 2 hour) • Runoff is overland and channel flow • Simplified channel routing • Negligible channel storage 	24-hour rainfall Rainfall distribution Runoff curve number Concentration time Drainage area
Unit Hydrograph (Gaged data) Synthetic Unit Hydrograph SCS Unit Hydrograph S-Graph Unit Hydrograph	<ul style="list-style-type: none"> • Midsize or large catchment (0.20 square miles to 1,000 square miles) • Uniformity of rainfall intensity and duration • Rainfall-runoff relationship is linear • Duration of direct runoff constant for all uniform-intensity storms of same duration, regardless of differences in the total volume of the direct runoff. • Time distribution of direct runoff from a given storm duration is independent of concurrent runoff from preceding storms • Channel-routing techniques used to connect streamflows 	Rainfall hyetograph and direct runoff hydrograph for one or more storm events Drainage area and lengths along main channel to point on watershed divide and opposite watershed centroid (Synthetic Unit Hydrograph)
Statistical (gage data) Log-Pearson Type III Bulletin #17B – U.S. Department of the Interior	<ul style="list-style-type: none"> • Midsized and large catchments with stream gage data • Appropriate station and/or generalized skew coefficient relationship applied • Channel storage 	10 or more years of gaged flood records
Basin Transfer of Gage Data	<ul style="list-style-type: none"> • Similar hydrologic characteristics • Channel storage 	Discharge and area for gaged watershed Area for ungaged watershed

* Magnitude and Frequency of Floods in California

Two other hydrologic software models that are commonly used are the Army Corps of Engineers' HEC-HMS and the National Resources Conservation Service's TR-20 Method.

The NOAA Atlas 14 product is the preferred IDF tool for State highway projects. See <http://hdsc.nws.noaa.gov/hdsc/pfds/>.

819.7 Region-Specific Analysis

(1) Desert Hydrology

Figure 819.7A shows the different desert regions in California, each with distinct hydrological characteristics that will be explained in this section.

(a) Storm Type

Summer Convective Storms - In the southern desert regions (Owens Valley/Mono Lake, Mojave Desert, Sonoran Desert and the Colorado Desert), the dominant storm type is the local thunderstorm, specifically summer convective storms. These storms are characterized by their short duration, over a relatively small area (generally less than 20 mi²), and intense rainfall, which may result in flash floods. These summer convective storms may occur at any time during the year, but are most common and intense during the summer. General summer storms can also occur over these desert regions, but are rare, and usually occur from mid-August to early October. The rainfall intensity can vary from heavy rainfall to heavy thunderstorms.

General Winter Storm - In the Antelope Valley and Northern Basin and Range regions, the dominant storm type is the general winter storm. These storms are characterized by their long duration, 6 hours to 12 hours or more, and possibly intermittently for 3 days to 5 days over a relatively large area. General winter storms produce the majority of large peaks in the northern desert areas; the majority of the largest peaks discharge greater than or equal to 20 cfs/mi² occurred during the winter and fall months in the Owens Valley/Mono Lake and Northern Basin and Range regions. At elevations above 6,000 ft, much

of the winter precipitation falls as snow; however, snowfall doesn't play a significant role in flood-producing runoff in the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert). In the northern desert regions (Owens Valley/Mono Lake and Northern Basin and Range), more floods from snowmelt occur at lower elevations; more than 50 percent of runoff events occurred in spring, most likely snowmelt, but did not produce large floods.

(b) Regional Regression

Newly developed equations for California's Desert regions are shown on Table 819.7A.

While the regression equations for the Northern Basin and Range region provide more accurate results than previous USGS developed equations, there is some uncertainty associated with them. Therefore, the development of a rainfall-runoff model may be preferable for ungaged watersheds in this region.

(c) Rational Method

The recommended upper limit for California's desert regions is 160 acres (0.25 mi²).

Table 819.7B lists common runoff coefficients for Desert Areas. These coefficients are applicable for storms with 2-year to 10-year return intervals, and should be adjusted for larger, less frequent storms by multiplying the coefficient by an appropriate frequency factor, C(f), as stated in Index 819.2(1) of this manual. The frequency factors, C(f), for 25-year, 50-year and 100-year storms are 1.1, 1.2 and 1.25, respectively. Under no circumstances should the product of C(f) times the runoff coefficient exceed 1.0. It is recommended not to use a value that exceeds 0.95.

(d) Rainfall-Runoff Simulation

A rainfall-runoff simulation approach uses a numerical model to simulate the rainfall-runoff process and generate discharge hydrographs. It has four main components:

rainfall; rainfall losses; transformation of effective rainfall; and channel routing.

(1) Rainfall

(a) Design Rainfall Criteria

The selection of an appropriate storm duration depends on a number of factors, including the size of the watershed, the type of rainfall-runoff approach and hydrologic characteristics of the study watershed. Watershed sizes are analyzed below and are applied to California's Desert regions in Table 819.7C.

Drainage Areas ≤ 20 mi² – Drainage areas less than 20 mi² are primarily representative of summer convective storms, and usually occur in the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert regions). Since these storms usually result in intense rainfall, over a small drainage area and are generally less than 6 hours, it is recommended that a 6-hour local design storm be utilized.

Drainage Areas > 20 mi² & ≤ 100 mi² – For drainage areas between 20 mi² and 100 mi², the critical storm can be a summer convective storm or a general thunderstorm. For these drainage areas, it is recommended that both 6-hour and 24-hour design storm be analyzed, and the storm that produces the largest peak discharge be chosen as the design basis.

Drainage Areas > 100 mi² – Since general storms usually cover a larger area and have a longer duration, for drainage areas greater than 100 mi², a 24-hour design storm is recommended.

(b) Depth-Duration-Frequency Characteristics

In 2011, NOAA published updated precipitation-frequency estimates for all of California including the desert regions, often cited as NOAA Atlas 14. This information is available online, via the Precipitation Frequency Data Server at <http://hdsc.nws.noaa.gov/hdsc/pfds/> NOAA Atlas 14 supersedes NOAA's previous effort, NOAA Atlas 2, the 2004 Atlas 14 which covered the Southwestern U.S., and California's Department of Water Resources (DWR) Bulletin No. 195, where their coverages overlap.

NOAA Atlas 14 provides a vast amount of information, which includes:

- Point Estimates
- ESRI shapefiles and ArcInfo ASCII grids
- Color cartographic maps: all possible combination of frequencies (2-year to 1,000-year) and durations (5-minute to 60-day)
- Associated Federal Geographic Data Committee-compliant metadata
- Data series used in the analysis: annual maximum series and partial duration series
- Temporal distributions of heavy precipitation (6-hour, 12-hour, 24-hour and 96-hour)
- Seasonal exceedance graphs: counts of events that exceed the 1 in 2, 5, 10, 25, 50 and 100 annual exceedance probabilities for the 60-minute, 24-hour, 48-hour and 10-day durations

Figure 819.7A
Desert Regions in California



Table 819.7A

Regional Regression Equations for California's Desert Regions

Region(s)	Associated Regression Equations
Colorado Desert Sonoran Desert Antelope Valley Mojave Desert	$Q_2 = 8.57A^{0.5668} \qquad Q_{25} = 291.04A^{0.5939}$ $Q_5 = 80.32A^{0.541} \qquad Q_{50} = 397.82A^{0.6189}$ $Q_{10} = 146.33A^{0.549} \qquad Q_{100} = 557.31A^{0.6619}$
Owens Valley / Mono Lake	$Q_2 = 0.007A^{1.839} \left[\frac{ELEV}{1000} \right]^{1.485} \left[\frac{LAT - 28}{10} \right]^{-0.680}$ $Q_5 = 0.212A^{1.404} \left[\frac{ELEV}{1000} \right]^{0.882} \left[\frac{LAT - 28}{10} \right]^{-0.030}$ $Q_{10} = 1.28A^{1.190} \left[\frac{ELEV}{1000} \right]^{0.531} \left[\frac{LAT - 28}{10} \right]^{0.525}$ $Q_{25} = 9.70A^{0.962} \left[\frac{ELEV}{1000} \right]^{0.107} \left[\frac{LAT - 28}{10} \right]^{1.199}$ $Q_{50} = 34.5A^{0.829} \left[\frac{ELEV}{1000} \right]^{-0.170} \left[\frac{LAT - 28}{10} \right]^{1.731}$ $Q_{100} = 111A^{0.707} \left[\frac{ELEV}{1000} \right]^{-0.429} \left[\frac{LAT - 28}{10} \right]^{2.241}$
Northern Basin & Range	$Q_2 = 5.320A^{0.415} \left[\frac{H}{1000} \right]^{0.928}$ $Q_5 = 29.71A^{0.360} \left[\frac{H}{1000} \right]^{0.296}$ $Q_{10} = 85.76A^{0.314} \left[\frac{H}{1000} \right]^{-0.109}$ $Q_{25} = 275.5A^{0.253} \left[\frac{H}{1000} \right]^{-0.555}$ $Q_{50} = 616.9A^{0.281} \left[\frac{H}{1000} \right]^{-0.867}$ $Q_{100} = 1293A^{0.166} \left[\frac{H}{1000} \right]^{-1.154}$

Table 819.7B**Runoff Coefficients for Desert Areas**

Type of Drainage Area	Runoff Coefficient
Undisturbed Natural Desert or Desert Landscaping (without impervious weed barrier)	0.30 – 0.40
Desert Landscaping (with impervious weed barrier)	0.55 – 0.85
Desert Hillslopes	0.40 – 0.55
Mountain Terrain (slopes greater than 10%)	0.60 – 0.80

Table 819.7C**Watershed Size for California Desert Regions**

Desert Region	Duration (based on Watershed size)
Southern Regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert)	6-hour local storm (≤ 20 mi ²)
	6-hour local storm and 24-hour general storm (between 20 mi ² & 100 mi ²); use the larger peak discharge
	24-hour general storm (> 100 mi ²)
Northern Regions (Owens Valley/Mono Lake and Northern Basin and Range)	24-hour general storm

(c) Depth-Area Reduction

Depth-area reduction is the method of applying point rainfall data from one or several gaged stations within a watershed to that entire watershed. NOAA Atlas 14 provides high resolution depth-duration frequency point data which can then be computed with other depth-duration frequency data in that cell to obtain an average depth-duration frequency over a watershed. However, as this data is available as point data, the average calculated depth-duration frequency may not represent an entire watershed. To convert this point data into watershed area, a conversion factor may be applied, of which, two methods are available: applying a reduction factor; or applying depth-area reduction curves.

NOAA is currently working on updating the reduction factors, thus, until then, the depth-area reduction curves are recommended. Two depth-area reduction curves are available: (1) the depth curves in National Weather Service's HYDRO-40

(http://www.nws.noaa.gov/oh/hdsc/PF_related_studies/TechnicalMemorandum_HYDRO40.pdf); and (2) the depth curves in NOAA Atlas 2. The general consensus is that the depth curves from HDRO-40 better represent the desert areas of California, and are recommended for the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and the Mojave Desert). For the upper regions (Owens Valley/Mono Lake and Northern Basin and Range), the curves from NOAA Atlas 2 are recommended.

The variables needed to apply depth area reduction curves to a

watershed are a storm frequency (i.e., a 100-year storm), storm duration (i.e., a 30-minutes storm), and the area of a watershed. For example, if a 100-year storm with a duration of 60-minutes were to be analyzed over a desert watershed of 25 mi², then using Figure 819.7B, the Depth-Area Ratio would be 0.64. This ratio would then be multiplied by the averaged point-rainfall data, which would then result in the rainfall over the entire watershed.

Point rainfall data is available from NOAA Atlas 14, which must then be converted to area rainfall data. Conversions are available in two forms: (1) the National Weather Service's HYDRO-40, and (2) NOAA Atlas 2. The National Weather Service's HYDRO-40 is recommended for the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert.) NOAA Atlas 2 is recommended for the northern desert regions (Owens Valley/Mono Lake and Northern Basin and Range).

(2) Rainfall Losses

Antecedent Moisture Condition – The Antecedent Moisture Condition (AMC) is the amount of moisture present in the soil before a rainfall event, or conversely, the amount of moisture the soil can absorb before becoming saturated (Note: the AMC is also referred to as the Antecedent Runoff Condition [ARC]). Once the soil is saturated, runoff will occur. Generally, the AMC is classified into three levels:

- AMC I – Lowest runoff potential. The watershed soils are dry enough to allow satisfactory grading or cultivation to take place.

- AMC II – Moderate runoff potential. AMC II represents an average study condition.
- AMC III – Highest runoff potential. The watershed is practically saturated from antecedent rainfall.

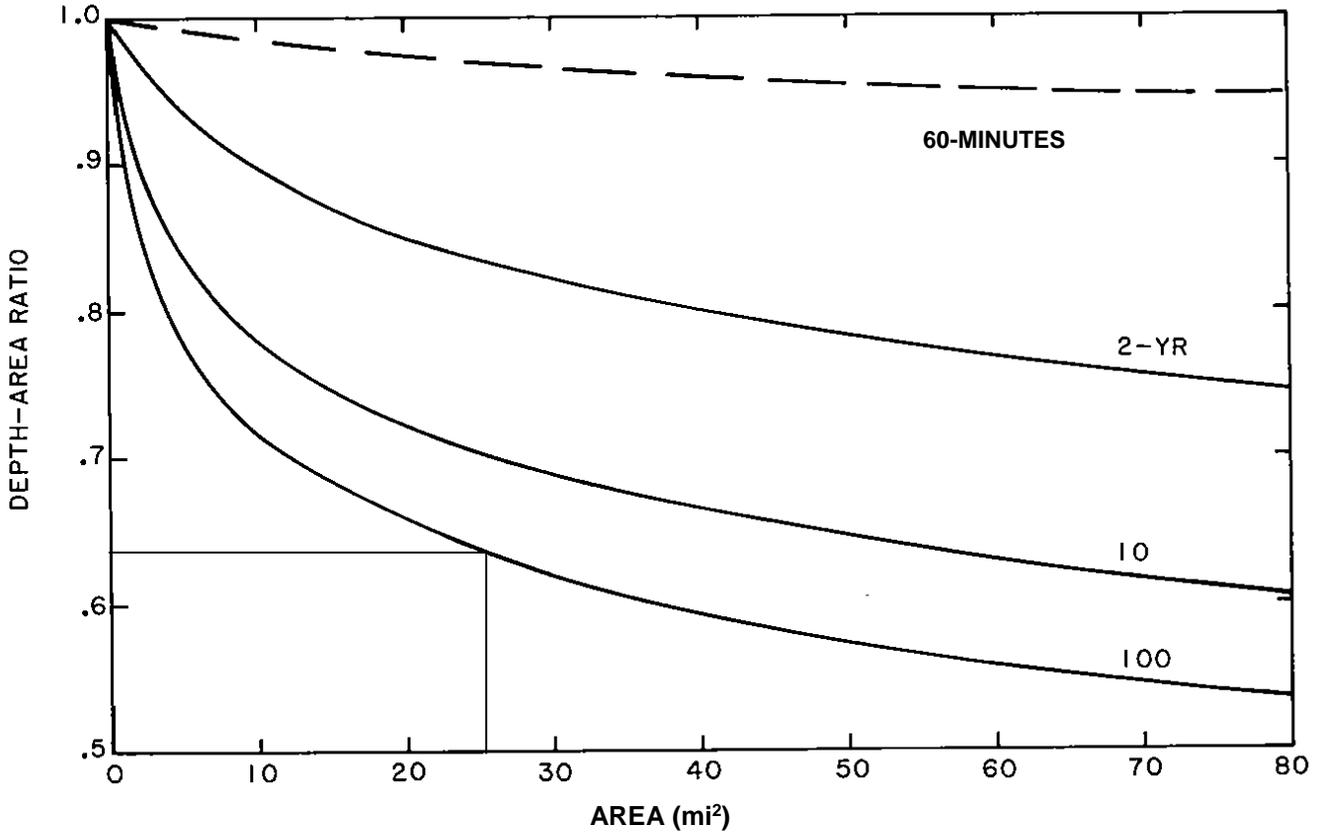
Because of the different storm types present in California's desert regions, AMC I is recommended as design criteria for local thunderstorms, and AMC II is recommended as design criteria for general storms.

Curve Number – The curve number was developed by the then Soil Conservation Service (SCS), which is now called the National Resource Conservation Service (NRCS). The curve number is a function of land use, soil type and the soil's AMC, and is used to describe a drainage area's storm water runoff potential. The soil type(s) are typically listed by name and can be obtained in the form of a soil survey from the local NRCS office. The soil surveys classify and present the soil types into 4 different hydrological groups, which are shown in Table 819.7D. From the hydrological groups, curve numbers are assigned for each possible land use-soil group combinations, as shown in Table 819.7E. The curve numbers shown in Table 819.7E are representative of AMC II, and need to be converted to represent AMC I, and AMC III, respectively. The following equations to convert an AMC II curve number to an AMC I or AMC III curve number, using a five-day period as the minimum for estimating the AMC's:

$$CN_{AMCI} = \frac{4.2CN_{AMCII}}{10 - 0.058CN_{AMCII}}$$

$$CN_{AMCIII} = \frac{23CN_{AMCII}}{10 + 0.13CN_{AMCII}}$$

Figure 819.7B
Example Depth-Area Reduction Curve



Note: The AMC of a storm area may vary during a storm; heavy rain falling on AMC I soil can change the AMC from I to II or III during the storm.

Table 819.7D
Hydrologic Soil Groups

Hydrologic Soil Group	Soil Group Characteristics
A	Soils having high infiltration rates, even when thoroughly wetted and consisting chiefly of deep, well to excessively-drained sands or gravels. These soils have a high rate of water transmission.
B	Soils having moderate infiltration rates when thoroughly wetted and consisting of moderately deep to deep, moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C	Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.
D	Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

(3) Transformation

Total runoff can be characterized by two types of runoff flow: direct runoff and base flow. Direct runoff is classified as storm runoff occurring during or shortly after a storm event. Base flow is classified as subsurface runoff from prior precipitation events and delayed subsurface runoff from the current storm. The transformation of precipitation runoff to excess can be accomplished using a unit hydrograph approach. The unit hydrograph method is based on the assumption that a watershed, in converting precipitation excess to runoff, acts as a linear, time-invariant system.

Unit Hydrograph Approach

A unit hydrograph for a drainage area is a curve showing the time distribution of runoff that would result at the concentration point from one inch of effective rainfall over the drainage area above that point.

The unit hydrograph method assumes that watershed discharge is related to the total volume of runoff, that the time factors that affect the unit hydrograph shape are invariant, and that watershed rainfall-runoff relationships are characterized by watershed area, slope and shape factors.

(a) SCS Unit Hydrograph

The SCS dimensionless unit hydrograph is based on averages of unit hydrographs derived from gaged rainfall and runoff for a large number of small rural basins throughout the U.S. The definition of the SCS unit hydrograph normally only requires one parameter, which is lag, defined as the time from the centroid of precipitation excess to the time of the peak of the unit hydrograph. For ungaged watersheds, the SCS suggests that the unit hydrograph lag time, t_{lag} , may be related to time

Table 819.7E**Curve Numbers for Land Use-Soil Combinations**

Description	Average % Impervious	Curve Number by Hydrological Soil Group				Typical Land Uses
		A	B	C	D	
Residential (High Density)	65	77	85	90	92	Multi-Family, Apartments, Condos, Trailer Parks
Residential (Medium Density)	30	57	72	81	86	Single-Family, Lot Size ¼ to 1 acre
Residential (Low Density)	15	48	66	78	83	Single-Family, Lot Size 1 acre or greater
Commercial	85	89	92	94	95	Strip Commercial, Shopping Centers, Convenience Stores
Industrial	72	81	88	91	93	Light Industrial, Schools, Prisons, Treatment Plants
Disturbed / Transitional	5	76	85	89	91	Gravel Parking, Quarries, Land Under Development
Agricultural	5	67	77	83	87	Cultivated Land, Row Crops, Broadcast Legumes
Open Land – Good	5	39	61	74	80	Parks, Golf Courses, Greenways, Grazed Pasture
Meadow	5	30	58	71	78	Hay Fields, Tall Grass, Ungrazed Pasture
Woods (Thick Cover)	5	30	55	70	77	Forest Litter and Brush adequately cover soil
Woods (Thin Cover)	5	43	65	76	82	Light Woods, Woods-Grass Combination, Tree Farms
Impervious	95	98	98	98	98	Paved Parking, Shopping Malls, Major Roadways
Water	100	100	100	100	100	Water Bodies, Lakes, Ponds, Wetlands

of concentration t_c , through the following relation:

$$t_{lag} = 0.6t_c$$

The time of concentration is the sum of travel time through sheet flow, shallow concentrated flow, and channel flow segments. A typical SCS Unit Hydrograph is similar to Figure 816.5.

A unit hydrograph can be derived from observed rainfall and runoff, however either may be unavailable. In such cases, a synthetic unit hydrograph can be developed using the S-graph method.

(b) S-graph

An S-graph is a summation hydrograph of runoff that would result from the continuous generation of unit storm effective rainfall over the area (1-inch per hour continuously). The S-graph method uses a basic time-runoff relationship for a watershed type in a form suitable for application to ungauged basins, and is based upon percent of ultimate discharge and percent of lag time. Several entities, including local and Federal agencies, have developed location-specific S-Graphs that are applicable to California's desert regions.

The ordinate is expressed in percent of ultimate discharge, and the abscissa is expressed in percent of lag time. Ultimate discharge, which is the maximum discharge attainable for a given intensity, occurs when the rate of runoff on the summation hydrograph reaches the rate of effective rainfall.

Lag for a watershed is an empirical expression of the hydrologic characteristics of a watershed in terms of time. It is defined as the elapsed time (in hours) from the

beginning of unit effective rainfall to the instant that the summation hydrograph for the point of concentration reaches 50 percent of ultimate discharge. When the lags determined from summation hydrographs for several gaged watersheds are correlated to the hydrologic characteristics of the watersheds, an empirical relationship is usually apparent. This relationship can then be used to determine the lags for comparable ungauged drainage areas for which the hydrologic characteristics can be determined, and a unit hydrograph applicable to the ungauged watersheds can be easily derived.

Figure 819.7C is a sample illustration of a San Bernardino County S-Graph, while Figure 819.7D shows an example S-Graph from USBR.

Recommendations

For watersheds with mountainous terrain/high elevations in the upper portions, the San Bernardino County Mountain S-Graph (<http://www.sbcounty.gov/dpw/floodcontrol/pdf/HydrologyManual.pdf>) is recommended. For watersheds in the southern desert regions with limited or no mountainous terrain/high elevations, the San Bernardino County Desert S-Graph (<http://www.sbcounty.gov/dpw/floodcontrol/pdf/HydrologyManual.pdf>) is recommended. The U.S. Bureau of Reclamation (USBR) S-Graph (http://www.usbr.gov/pmts/hydraulics_lab/pubs/manuals/SmallDams.pdf) is recommended for watersheds in the Northern Basin and Range.

As an alternative to the above mentioned S-Graphs, the SCS Unit Hydrograph may also be used.

Figure 819.7C
San Bernardino County Hydrograph for Desert Areas

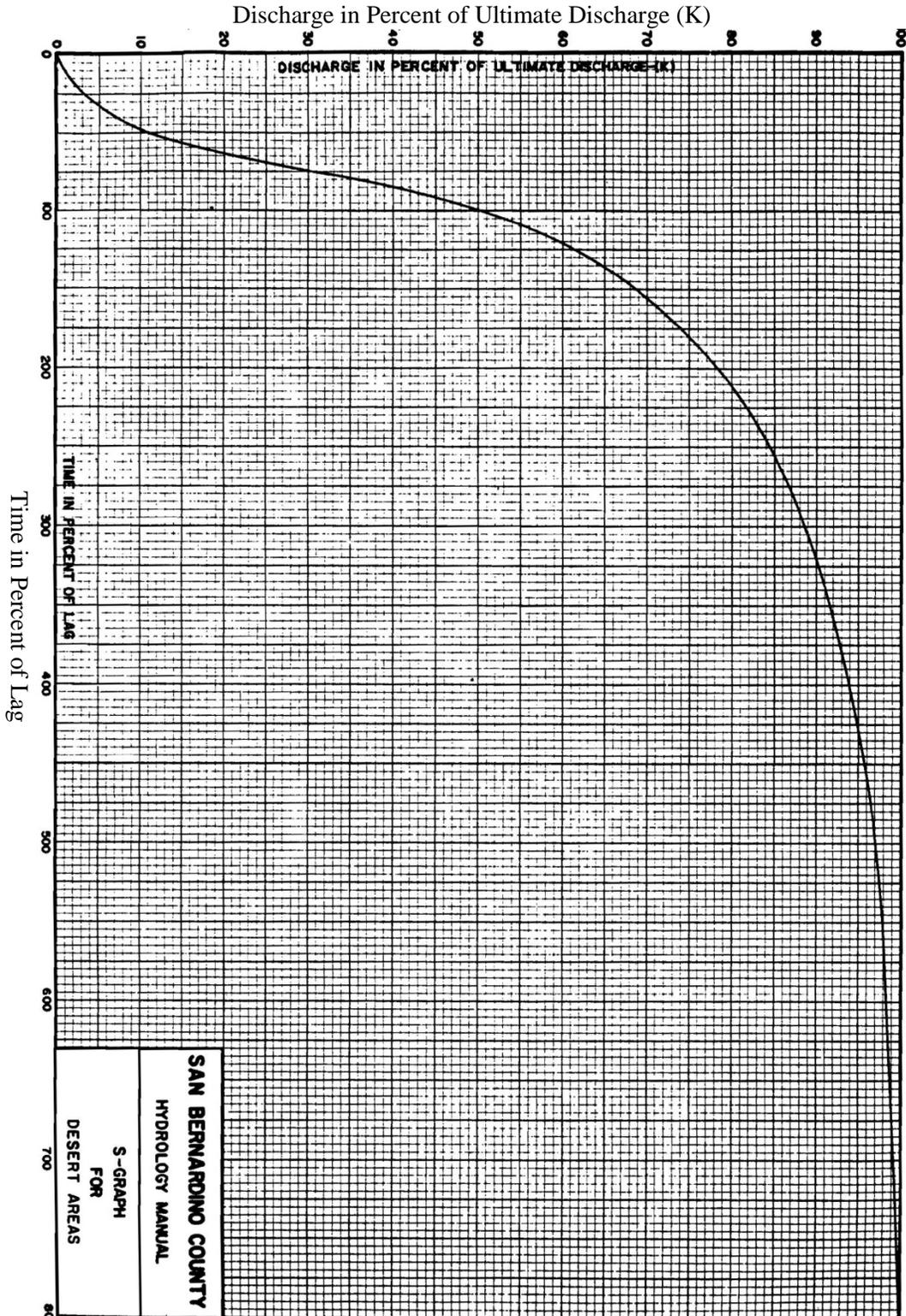


Figure 819.7D
USBR Example S-Graph

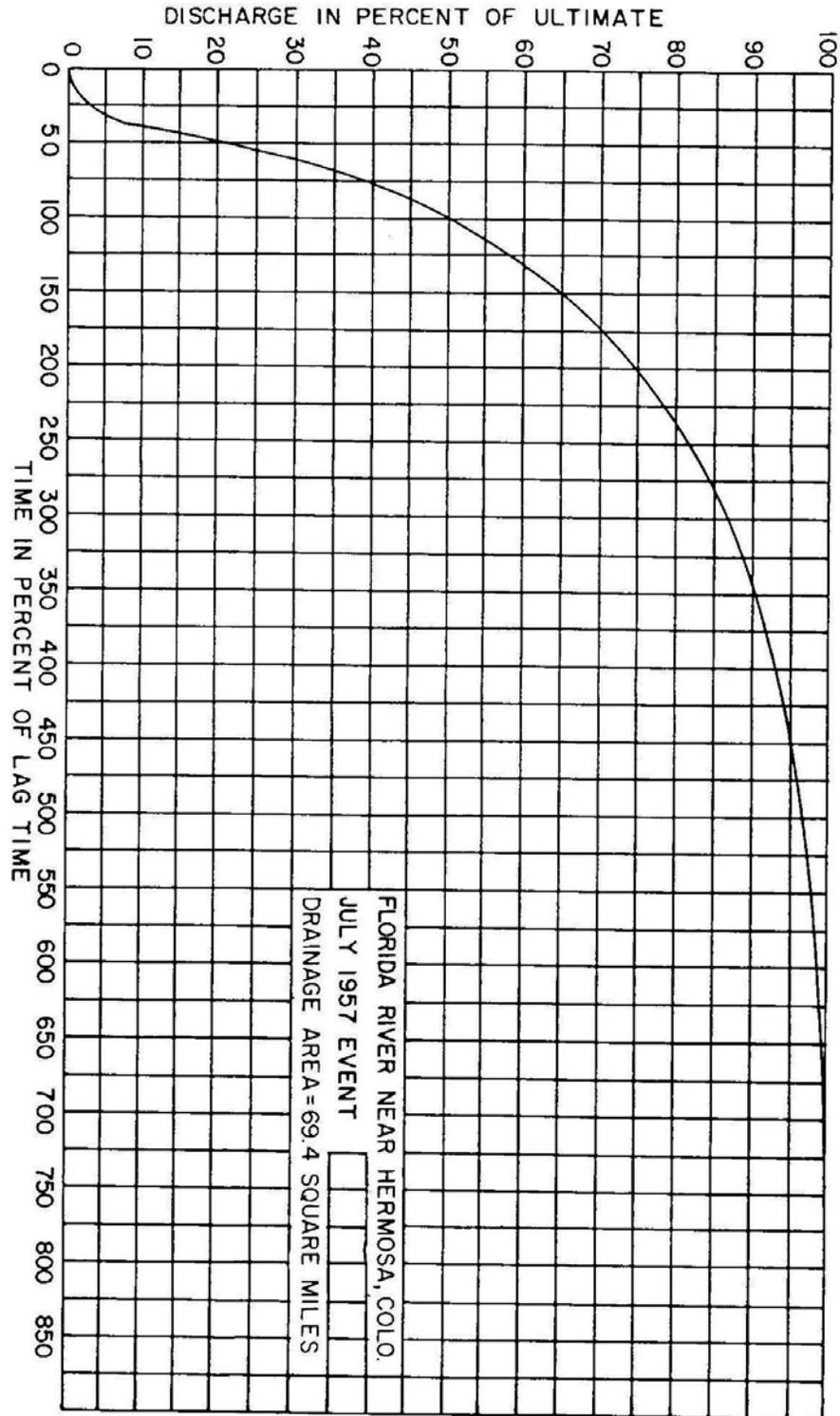


Table 819.7F
Channel Routing Methods

Routing Method	Pros	Cons
Kinematic Wave	<ul style="list-style-type: none"> ▪ A conceptual model assuming a uniform flow condition. ▪ In general, works best for steep (10 ft/mile or greater), well defined channels. ▪ It is often applied in urban areas because the routing reaches are generally short and well-defined. 	<ul style="list-style-type: none"> ▪ Cannot handle hydrograph attenuation, significant overbank storage, and backwater effects.
Modified Puls	<ul style="list-style-type: none"> ▪ Known as storage routing or level-pool routing. ▪ Can handle backwater effects through the storage-discharge relationship. 	<ul style="list-style-type: none"> ▪ Need to use hydraulic model to define the required storage-outflow relationship.
Muskingum	<ul style="list-style-type: none"> ▪ Directly accommodates the looped relationship between storage and outflow. ▪ A linear routing technique that uses coefficients to account for hydrograph timing and diffusion. 	<ul style="list-style-type: none"> ▪ The coefficients cannot be used to model a range of floods that may remain in bank or go out of bank. Therefore, not applicable to significant overbank flows.
Muskingum-Cunge	<ul style="list-style-type: none"> ▪ A nonlinear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. ▪ The parameters are physically based. ▪ Has been shown to compare well against the full unsteady flow equations over a wide range of flow conditions. 	<ul style="list-style-type: none"> ▪ It cannot account for backwater effects. ▪ Not very applicable for routing a very rapidly rising hydrograph through a flat channel.

(4) Channel Routing

Channel routing is a process used to predict the temporal and spatial variation of a flood hydrograph as it moves through a river reach. The effects of storage and flow resistance within a river reach are reflected by changes in hydrograph shape and timing as the flood wave moves from upstream to downstream. The four commonly used methods are the kinematic wave routing, Modified Puls routing, Muskingum routing, and Muskingum-Cunge routing. The advantages and disadvantages for each method are described in Table 819.7F. Table 819.7G provides guidance for selecting an appropriate routing method. The Muskingum-Cunge routing method can handle a wide range of flow conditions with the exception of significant backwater. The Modified Puls routing can model backwater effects. The kinematic wave routing method is often applied in urban areas with well defined channels.

(5) Storm Duration and Temporal Distribution

Temporal distribution is the time-related distribution of the precipitation depth within the duration of the design storm. Temporal distribution patterns of design storms are based on the storm duration. The temporal distribution pattern for short-duration storms represents a single cloudburst and is based on rainfall statistics. The temporal distribution for long-duration storms resembles multiple events and is patterned after historic events. Since the storm events in California’s desert regions are made up of two distinct separate storm types, the summer convective storm and the general winter storm, the design storm durations should be adjusted accordingly. For California’s desert regions, the 100-year 6-hour storm is recommended for the convective storms, and the 100-year 24-

hour storm is recommended for the winter storms. Table 819.7H summarizes the design storm durations for the different desert regions throughout California.

**Table 819.7G
Channel Method Routing
Guidance**

IF THIS IS TRUE...	... THEN THIS ROUTING MODEL MAY BE CONSIDERED.
No observed hydrograph data available for calibration	Kinematic wave; Muskingum-Cunge
Significant backwater will influence discharge hydrograph	Modified Puls
Flood wave will go out of bank, into floodplain.	Modified Puls; Muskingum-Cunge with 8-point cross section
Channel slope > 0.002 and $\frac{TS_o u_o}{d_o} \geq 171$	Any
Channel slopes from 0.002 to 0.0004 and $\frac{TS_o u_o}{d_o} \geq 171$	Muskingum-Cunge; Modified Puls; Muskingum
Channel slope < 0.0004 and $TS_o \left(\frac{g}{d_o}\right)^{1/2} \geq 30$	Muskingum-Cunge
Channel slope < 0.0004 and $TS_o \left(\frac{g}{d_o}\right)^{1/2} < 30$	None

Notes:

- T = hydrograph duration
- u_o = reference mean velocity
- d_o = reference flow depth
- S_o = channel slope

(2) Sediment/Debris Bulking

The process of increasing the water volume flow rate to account for high concentrations of sediment and debris is defined as bulking. Debris carried in the flow can be significant and greatly increase flow volume conveyed from a watershed. This condition occurs frequently in mountainous areas subject to wildfires with soil erosion, as well as arid regions around alluvial fans and other geologic activity. By bulking the flow through the use of an appropriate bulking factor, bridge openings and culverts can be properly sized for areas that experience high sediment and debris concentration.

(a) Bulking Factor

Bulking factors are applied to a peak (clear-water) flow to obtain a total or bulked peak flow, which provides a safety factor in the sizing of hydraulic structures. For a given watershed, a bulking factor is typically a function of the historical concentration of sediment in the flow.

(b) Types of Sediment/Water Flow

The behavior of flood flows will vary depending on the concentration of sediment in the mixed flow, where the common flow types are normal stream flow, hyperconcentrated flow, and debris flow.

(1) Normal Stream Flow

During normal stream flow, the sediment load minimally influences flow behavior or characteristics. Because sediment has little impact, this type of flow can be analyzed as a Newtonian fluid and standard hydraulic methods can be used. The upper limit of sediment concentration by volume for normal stream flow is 20 percent and bulking factors are applied cautiously because of the low concentration. (See Table 819.7I) The small amount of sediment is conveyed by conventional suspended load and bed-load.

(2) Hyperconcentrated Flow

Hyperconcentrated flow is more commonly known as mud flow. Because of potential for large volumes of sand in the water column, fluid properties and transport characteristics change and the mixture does not behave as a Newtonian fluid. However, basic hydraulic methods and models are still generally accepted and used for up to 40 percent sediment concentration by volume. For hyperconcentrated flow, bulking factors vary between 1.43 and 1.67 as shown in Table 819.7I.

(3) Debris Flow

In debris flow state, behavior is primarily controlled by the composition of the sediment and debris mixture, where the volume of clay can have a strong influence in the yield strength of the mixture.

During debris flow, which has an upper limit of 50 percent sediment concentration by volume, the sediment/debris/water mixture no longer acts as a Newtonian fluid and basic hydraulic equations do not apply. If detailed hydraulic analysis or modeling of a stream operating under debris flow is needed, FLO2DH is the recommended software choice given its specific debris flow capabilities. HEC-RAS is appropriate for normal stream flow and hyperconcentrated flow, but cannot be applied to debris flow.

For a typical debris flow event, clear-water flow occurs first, followed by a frontal wave of mud and debris. Low frequency events, such as the 100-year flood, most likely contain too much water to produce a debris flow event. Normally, smaller higher frequency events such as 10-year or 25-year floods actually have a greater probability of yielding a debris flow event requiring a higher bulking factor.

As outlined in Table 819.7I, bulking factors for debris flow vary between 1.67 and 2.00.

(c) Sediment/Debris Flow Potential

(1) Debris Hazard Areas

Mass movement of rock, debris, and soil is the main source of bulked flows. This can occur in the form of falls, slides, or flows. The volume of sediment and debris from mass movement can enter streams depending upon hydrologic and geologic conditions.

The location of these debris-flow hazards include:

- (a) At or near the toe of slope 2:1 or steeper
- (b) At or near the intersection of ravines and canyons
- (c) Near or within alluvial fans
- (d) Soil Slips

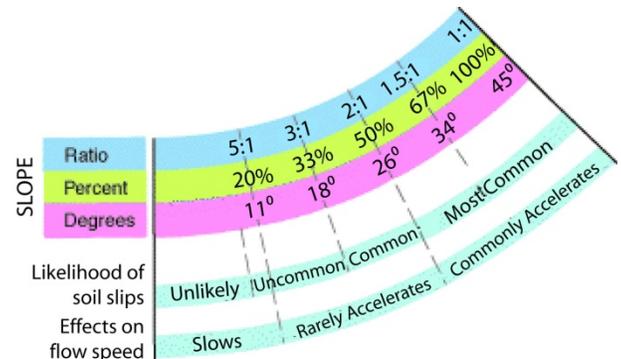
Soil slips commonly occur at toes of slope between 2:1 and 3:1. Flowing mud and rocks will accelerate down a slope until the flow path flattens. Once energy loss occurs, rock, mud, and vegetation will be deposited. Debris flow triggered by soil slips can become channelized and travel distances of a mile or more. Figure 819.7E shows the potential of soil slip versus slope angle. As seen in this Figure, the flatter the slope angle, the less effect on flow speed and acceleration.

(2) Geologic Conditions

In the Transverse Ranges that include the San Gabriel and San Bernardino Mountains along the southern and southwestern borders of the Antelope Valley (Region 3) and Mojave Desert (Region 4), their substrate contains sedimentary rocks, fractured basement rocks, and granitic rocks. This type of geology has a high potential of debris flow from the hillsides of these regions.

Figure 819.7E

Soil Slips vs. Slope Angle



While debris flow potential is less prevalent, it is possible to have this condition in the Peninsula Ranges that include the San Jacinto, Santa Rosa, and Laguna Mountains along the western border of the Colorado Desert (Region 1).

(d) Alluvial Fans

An alluvial fan is a landform located at the mouth of a canyon, formed in the shape of a fan, and created over time by deposition of alluvium. With the apex of the fan at the mouth of a canyon, the base of the fan is spread across lower lying plains below the apex. Over time, alluvial fans change and evolve when sediment conveyed by flood flows or debris flows is deposited in active channels, which creates a new channel within the fan. Potentially, alluvial fan flood and debris flows travel at high velocity, where large volumes of sediment can be eroded from mountain canyons down to the lower fan surface. Given this situation, the alignments of the active channels and the overall footprint of an alluvial fan are dynamic. Also, the concentration of sediment/debris volume is dynamic, ranging from negligible to 50 percent.

Alluvial fans can be found on soil maps, geologic maps, topographic maps, and aerial photographs, in addition to the best

Table 819.7H
Design Storm Durations

Drainage Area	Desert Region	100-year, 6-hour Convective Storm (AMC I)	100-year, 24-hour General Storm (AMC II)	Regional Regression Equations
< 20 mi ²	Colorado Desert	X		
	Sonoran Desert	X		
	Mojave Desert	X		
	Antelope Valley Desert	X		
> 20 mi ²	Colorado Desert	X*	X*	
	Sonoran Desert	X*	X*	
	Mojave Desert	X*	X*	
	Antelope Valley Desert	X*	X*	
	Owens Valley/Mono Lake			X**
	Northern Basin & Range		X	

* For watersheds greater than 20 mi² in the southern desert regions, both the 6-hour Convective Storm (AMC I) and the 24-hour General Storm (AMC II) should be analyzed and the larger of the two peak discharges selected.

** The use of regional regression equations is recommended where streamgage data are not available; otherwise, hydrologic modeling could be performed with snowmelt simulation.

Table 819.7I**Bulking Factors & Types of Sediment Flow**

Sediment Flow Type	Bulking Factor	Sediment Concentration by Weight	Sediment Concentration by Volume
		(100% by WT = 1×10^6 ppm)	(specific gravity = 2.65)
Normal Streamflow	0	0	0
	1.11	23	10
	1.25	40	20
Hyperconcentrated Flow	1.43	52	30
	1.67	53	40
Debris Flow	2.00	72	50
Landslide	2.50	80	60
	3.33	87	70

source which is a site visit. An example of an alluvial fan, shown in plan view, is in Figure 819.7F and Figure 872.3.

Figure 819.7F

Alluvial Fan



(e) Wildfire and Debris Flow

After fires have impacted a watershed, sediment/debris flows are caused by surface erosion from rainfall runoff and landsliding due to rainfall infiltration into the soil. The most dominant cause is the runoff process because fire generally reduces the infiltration and storage capacity of soils, which increases runoff and erosion.

(1) Fire Impacts

Arid regions do not have the same density of trees and vegetation as a forested area, but the arid environment still falls victim to fires in a similar manner. Prior to a fire, the arid region floor can contain a litter layer (leaves, needles, fine twigs, etc.), as well as a duff layer (partially decomposed components of the litter layer). These layers absorb water, provide storage of rainfall, and protect hillsides. Once

these layers are burned, they become ash and charcoal particles that seal soil pores and decrease infiltration potential of the soil, which ultimately increases runoff and erosion.

In order to measure the burn severity of watersheds with respect to hydrologic function, classes of burn severity have been created. These classes are simply stated as high, moderate, low, and unburned. From moderate and high burn severity slopes, the generated sediment can reach channels and streams causing bulked water flows during storm events. Generally speaking, the denser the vegetation in a watershed prior to a fire and the longer a fire burns within this watershed, the greater the effects on soil hydrologic function. This occurs due to the fire creating a water repellent layer at or near the soil surface, the loss of soil structural stability, which all results in more runoff and erosion. After a one or two-year period, the water repellent layer is usually washed away.

(f) Local Agency Methods For Predicting Bulking Factors

(1) San Bernardino County

Instead of conducting a detailed analysis, San Bernardino Flood Control District uses a set value for bulking of 2 (i.e., 100 percent bulking) for any project where bulking flows may be anticipated. This bulking factor of 2 can also be expressed as a 50 percent sediment concentration by volume, which is about the upper limit of debris flow. A higher percentage of sediment concentration would be considered a landslide instead of debris flow. Basically, the San Bernardino County method assumes debris flow conditions for all types of potential bulking.

(2) Los Angeles County

The Los Angeles (LA) County method uses a watershed-specific bulking factor. The LA County Sedimentation

Manual, which is located at <http://ladpw.org/wrd/publication>, divides the county into three basins: LA Basin, Santa Clara River Basin, and Antelope Valley, where only the latter is located in the Caltrans desert hydrology regions. The production of sediment from these basins is dependent upon many factors, including rainfall intensity, vegetative cover, and watershed slope. For each of the LA County basins, Debris Potential Area (DPA) zones have been identified.

The Design Debris Event (DDE) is associated with the 50-year, 24-hour duration storm, and produces the quantity of sediment from a saturated watershed that is recovered from a burn. For example, a DPA 1 zone sediment rate of 120,000 cubic yards per square mile has been established as the DDE for a 1-square mile drainage area. This sediment rate is recommended for areas of high relief and granitic formation found in the San Gabriel Mountains. In other mountainous areas in LA County, lower sediment rates have been assigned based on differences in topography, geology, and precipitation. For the Antelope Valley basin, eight debris production curves have been generated, and can be found in Appendix B of the LA County Sedimentation Manual along with curves for the other basins.

In addition to sediment production rates, a series of peak bulking factor curves are presented for each LA County basin in Appendix B of the LA manual. The peak bulking factor can be estimated using these curves based on the watershed area and the DPA. Within the Antelope Valley basin, maximum peak bulking factors range from 1.2 in DPA Zone 11 to 2.00 in DPA Zone 1.

(3) Riverside County

For Riverside County, a bulking factor is calculated by estimating a

sediment/debris yield rate for a specific storm event, and relating it to the largest expected sediment yield of 120,000 cubic yards per square mile for a 1-square mile watershed from the LA County procedure. This sediment rate from LA County is based on the DPA Zone 1 corresponding to the highest expected bulking factor of 2.00.

The bulking factor equation from the Riverside County Hydrology Manual (<http://www.floodcontrol.co.riverside.ca.us/downloads/planning/>) is as follows:

$$BF = 1 + \frac{D}{120,000}$$

BF = Bulking Factor

D = Design Storm Sediment/Debris Production Rate For Study Watershed (cubic yards/square mile)

(4) U.S. Army Corps of Engineers- LA District

This method, located at <http://www.spl.usace.army.mil/resreg/htdocs/Publications.html>, was originally developed to calculate unit sediment/debris yield values for an “n-year” flood event, and applied to the design and analysis of debris catching structures in coastal Southern California watersheds. The LA District method considers frequency of wildfires and flood magnitude in its calculation of unit debris yield. Even though its original application was intended for coastal-draining watersheds, this method can also be used for desert-draining watersheds for the same local mountain ranges.

The LA District method can be applied to watershed areas between 0.1 and 200 mi² that have a high proportion of their total area in steep, mountainous topography. This method is best used for watersheds that have received significant antecedent rainfall of at least 2 inches in 48 hours. Given this

criteria, the LA District method is more suited for general storms rather than thunderstorms.

As shown below, this method specifies a few equations to estimate unit debris yield dependent upon the areal size of the watershed. These equations were developed by multiple regression analysis using known sediment/debris data.

For watersheds between 3 and 10 mi², the following equations can be used:

$$\log Dy = 0.85 \log Q + 0.53 \log RR \\ + 0.04 \log A + 0.22 FF$$

D_y = Unit Debris Yield (cubic yards/square mile)

RR = Relief Ratio (foot/mile), which is the difference in elevation between the highest and lowest points on the longest watercourse divided by the length of the longest watercourse

A = Drainage Area (acres)

FF = Fire Factor

Q = Unit Peak Runoff (cfs/square mile)

In order to account for increase in debris yield due to fire, a non-dimensional fire factor (FF) is a component in the equation above. The FF varies from 3.0 to 6.5, with a higher factor indicating a more recent fire and more debris yield. This factor is 3.0 for desert watersheds because the threat and effects from fire are minimal.

Because the data used to develop the regression equation was taken from the San Gabriel Mountains, an Adjustment and Transposition (A-T) factor needs to be applied to debris yields from the study watersheds. The A-T factor can be determined using Table 819.7J by finding the appropriate subfactor for each of the four groups (Parent Material, Soils, Channel Morphology, and Hillside Morphology) and summing

the subfactors. This sum is the total A-T factor, and it must be multiplied by the sediment/debris yield.

Once the sediment/debris yield value has been determined based on the unit yield, a bulking factor can be calculated using a series of equations. The first equation provides a translation of the clear-water discharge to a sediment discharge. This clear-water discharge should be developed using a hydrograph method and a hydrologic modeling program, such as HEC-HMS.

$$Q_s = aQ_w^n$$

Q_s = Sediment Discharge (cfs)

Q_w = 100-Year Clear-Water Discharge (cfs)

a = Bulking Constant

For a majority of sand-bed streams, the value of "n" is between 2 and 3. When $n=2$, the bulking factor is linearly proportional to the clear-water discharge. As for the coefficient "a", it is determined with the following equation:

$$a = \frac{V_s}{\Delta t \sum Q_w^2}$$

V_s = Total Sediment Volume (cubic feet)

Δt = Computation Time Interval Used In Developing Hydrograph From Hydrologic Model (e.g. HEC-HMS)

Finally, the bulking factor equation is expressed as follows:

$$BF = \frac{Q_w - Q_s}{Q_s} = 1 + aQ_w^{n-1}$$

(g) Recommended Approach For Developing Bulking Factors

A flow chart outlining the recommended bulking factor process is provided in Figure 819.7H, which considers all bulking methods presented in Topic 819.

As shown in Steps 4 and 5 on Figure 819.7H, a bulking factor can be found by:

- (1) Identifying the type of flow within a watershed and selecting the corresponding bulking factor, or
- (2) Using one of the agency methods to calculate the bulking factor.

If the type of flow cannot be identified or the project site does not fall within the recommended boundaries from Figure 819.7H, use the LA District Method because it is the most universal given its use of the Adjustment-Transposition factor based on study watershed properties.

Table 819.7J

Adjustment-Transportation Factor Table

	A-T SUBFACTOR				
	0.25	0.20	0.15	0.10	0.05
PARENT MATERIAL	SUBFACTOR GROUP 1				
Folding	Severe		Moderate		Minor
Faulting	Severe		Moderate		Minor
Fracturing	Severe		Moderate		Minor
Weathering	Severe		Moderate		Minor
SOILS	SUBFACTOR GROUP 2				
Soils	Non-cohesive		Partly Cohesive		Highly Cohesive
Soil Profile	Minimal Soil Profile		Some Soil Profile		Well-developed Soil Profile
Soil Cover	Much Bare Soil in Evidence		Some Bare Soil in Evidence		Little Bare Soil in Evidence
Clay Colloids	Few Clay Colloids		Some Clay Colloids		Many Clay Colloids
CHANNEL MORPHOLOGY	SUBFACTOR GROUP 3				
Bedrock Exposures	Few Segments in Bedrock		Some Segments in Bedrock		Many Segments in Bedrock
Bank Erosion	> 30% of Banks Eroding		10 – 30% of Banks Eroding		< 10% of Banks Eroding
Bed and Bank Materials	Non-cohesive Bed and Banks		Partly Cohesive Bed and Banks		Mildly Cohesive Bed and Banks
Vegetation	Poorly Vegetated		Some Vegetation		Much Vegetation
Headcutting	Many Headcuts		Few Headcuts		No Headcutting
HILLSLOPE MORPHOLOGY	SUBFACTOR GROUP 4				
Rills and Gullies	Many and Active		Some Signs		Few Signs
Mass Movement	Many Scars Evident		Few Signs Evident		No Signs Evident
Debris Deposits	Many Eroding Deposits		Some Eroding Deposits		Few Eroding Deposits
The A-T Factor is the sum of the A-T Subfactors from all 4 Subfactor Groups.					

Figure 819.7H

Recommended Bulking Factor Selection Process

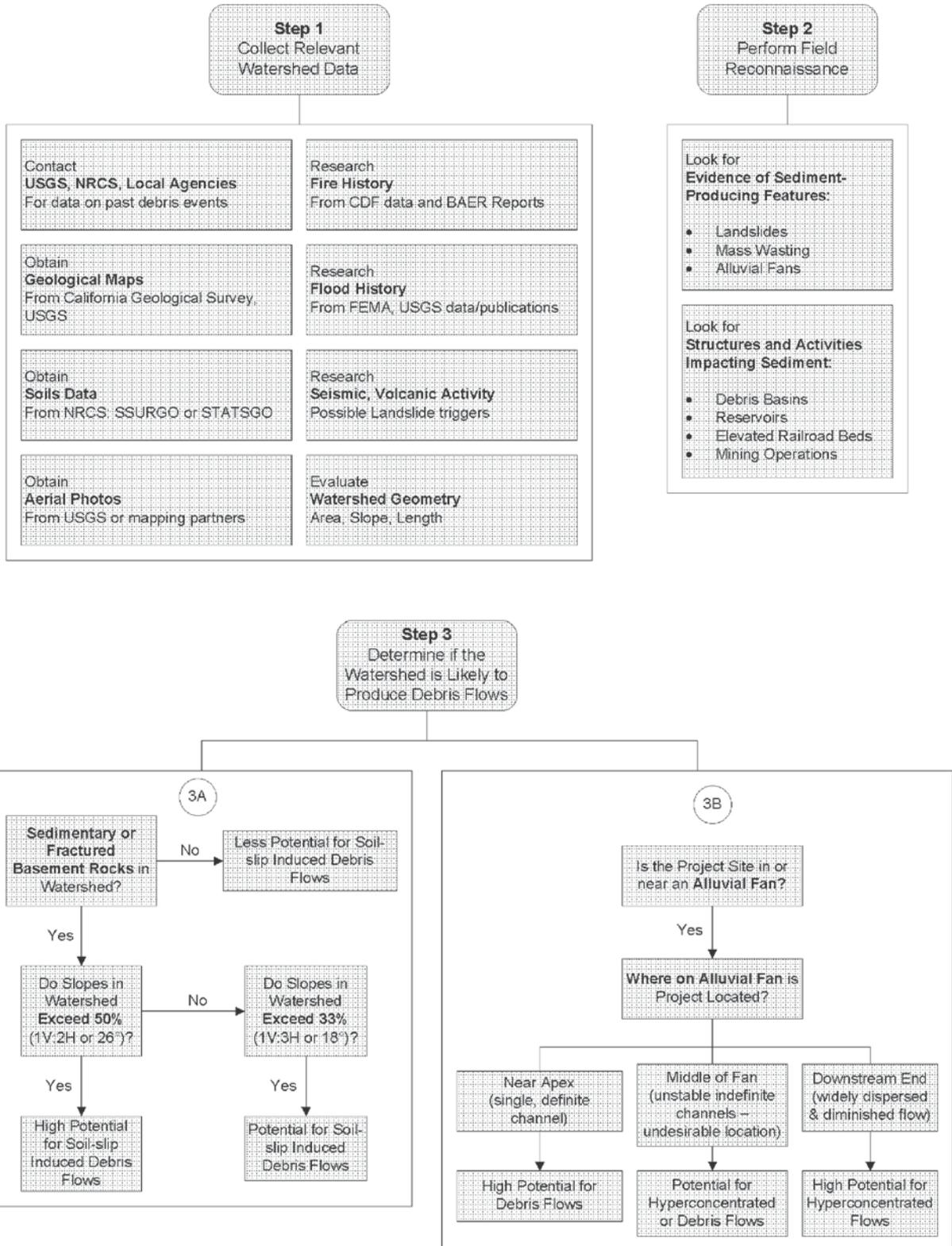
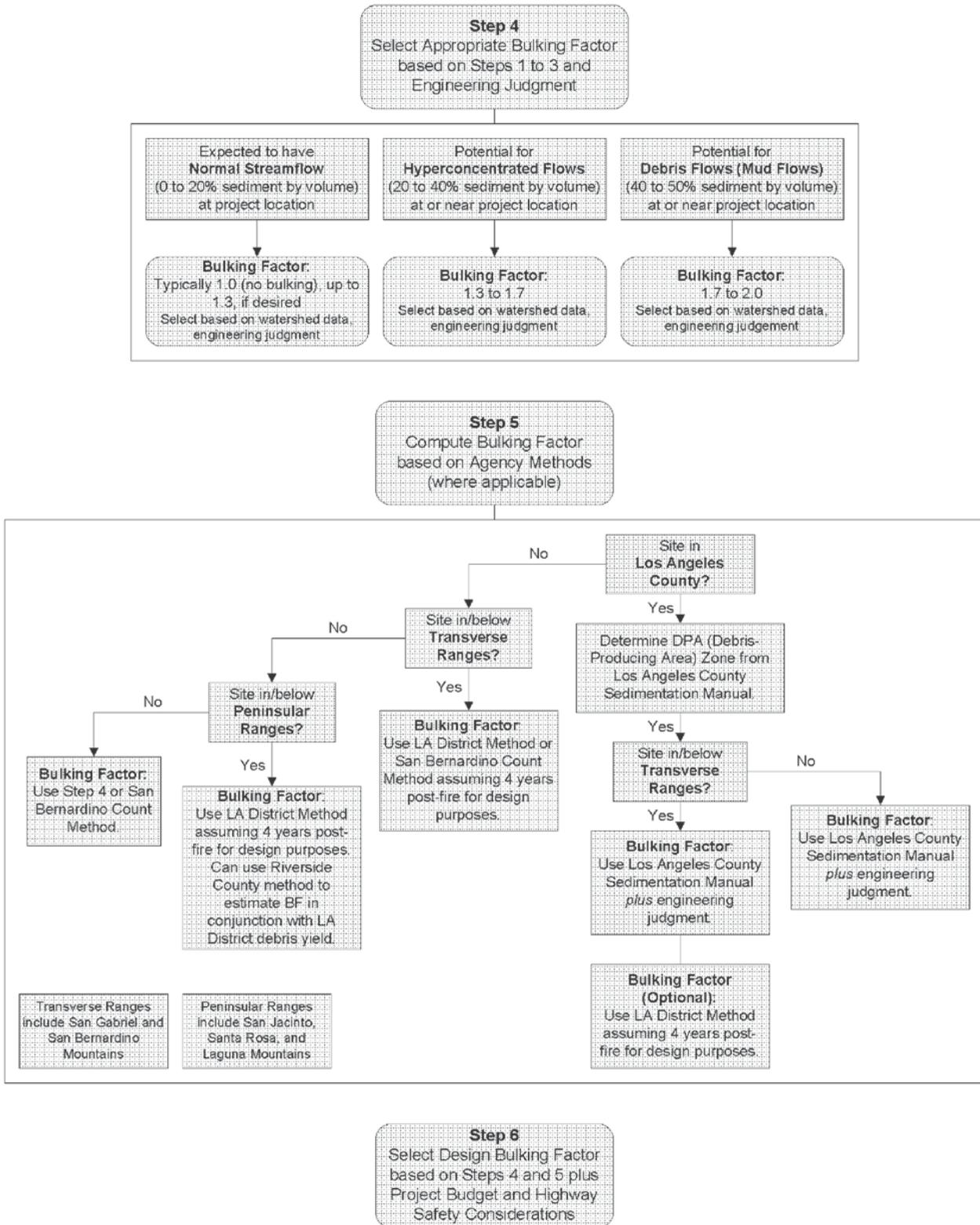


Figure 819.7H
Recommended Bulking Factor Selection Process (Cont'd)



objectionable velocities resulting in abrasion of the culvert itself or in downstream erosion. In most cases, provided the culvert is not flowing under pressure, an increase in the culvert size does not appreciably change the outlet velocities.

- (2) *Tailwater.* The term, tailwater, refers to the water located just downstream from a structure. Its depth or height is dependent upon the downstream topography and other influences. High tailwater could submerge the culvert outlet.

821.5 Effects of Tide, Storm Surge and Wind

Culvert outfalls and bridge openings located where they may be influenced by ocean tides require special attention to adequately describe the 1% probability of exceedance event.

Detailed statistical analysis and use of unsteady flow models, including two-dimensional models, provide the most accurate approach to describing the combined effects of tidal and meteorological events. Such special studies are likely warranted for major hydraulic structures (See HEC-25, "Highways in the Coastal Environment"), but would typically be too costly and time consuming for lesser facilities. Fortunately, for many situations, this detailed analysis already exists in the form of FEMA hydraulic models which include tidal impacts at stream/ocean confluences.

For all situations, the following guidelines are recommended:

(1) *Bridges*

- (a) If available, use information contained in FEMA hydraulic studies.
- (b) If FEMA models/studies are not available, conduct site specific analysis of tidal data in conjunction with meteorological storm data to arrive at the exceedance probability necessary for design (See Index 821.3).

(2) *Culverts*

- (a) If available, use information contained in FEMA hydraulic studies.

- (b) If FEMA models/studies are not available, base design on the more severe of the two following conditions:

- Q_{100} flood event combined with a condition of mean sea level, or
- Q_2 flood event combined with a condition of Design High Tide (See Figure 873.2A)

See Index 814.5 for resources on tide data. Tidal data includes the influence of storm surge as part of the tidal record, but does not account for waves, run-up or other wind driven elements that could impact structures. From a conveyance perspective, waves and run-up are not a consideration, but should be considered for their respective impacts to the physical integrity of the drainage structure and for potential operational impacts to the highway.

Topic 822 - Debris Control

822.1 Introduction

Debris, if allowed to accumulate either within a culvert or at its inlet, can adversely affect the hydraulic performance of the facility. Damage to the roadway and to upstream property may result from debris obstructing the flow into the culvert. Coordination with district maintenance forces can help in identifying areas with high debris potential and in setting requirements for debris removal where necessary.

The use of any device that can trap debris must be thoroughly examined prior to its use. In addition to the more common problem of debris accumulation at the culvert entrance, the use of safety end grates or other appurtenances can also lead to debris accumulation within the culvert at the outlet end. Evaluation of this possibility, and appropriate preventive action, must be made if such end treatment is proposed.

822.2 Debris Control Methods

There are two methods of handling debris:

- (1) *Passing Through Culvert.* If economically feasible, culverts should be designed to pass debris. Culverts which pass debris often have a higher construction cost. On the other hand,

retaining solids upstream from the entrance by means of a debris control structure often involves substantial maintenance cost and could negatively affect fish passage. An economic comparison which includes evaluation of long term maintenance costs should be made to determine the most reasonable and cost effective method of handling.

- (2) *Interception.* If it is not economical to pass debris, it should be retained upstream from the entrance by means of a debris control structure or the use of a debris basin when the facility is located in the vicinity of alluvial fans.

If drift and debris are retained upstream, a riser or chimney may be required. This is a vertical extension to the culvert which provides relief when the main entrance is plugged. The increased head should not be allowed to develop excessive velocities or cause pressure which might induce leakage in the culvert.

If debris control structures are used, access must be provided for maintenance equipment to reach the site. This can best be handled by coordination and field review with district maintenance staff. Details of a pipe riser with debris rack cage are shown on Standard Plan D93C. See FHWA Hydraulic Engineering Circular No. 9, "Debris-Control Structures" for further information.

The use of an upstream debris basin and downstream concrete lined channels, has often been used by Local Agencies for managing flood flows on alluvial fans in urbanized areas. Experience has shown that this approach is effective, however, the costs of building and maintaining such facilities is high with a potential for sediment inflows greater than anticipated.

The District Hydraulics Engineer should be consulted if a debris basin is being considered for interception in the vicinity of an alluvial fan.

822.3 Economics

Debris problems do not occur at all suspected locations. It is often more economical to construct

debris control structures after problems develop. An assessment of potential damage due to debris clogging if protection is not provided should be the basis of design.

822.4 Classification of Debris

In order to properly determine methods for debris control, an evaluation of the characteristics of debris within flood flows must be made. Debris can be either floating, suspended in the flood flow, or dragged/rolled along the channel bottom. Typically, a flood event will deposit debris from all of these types.

The FHWA Hydraulic Engineering Circular No. 9 contains a debris classification system to aid the designer in selecting the appropriate type of debris control structure.

822.5 Types of Debris Control Structures

The FHWA Hydraulic Engineering Circular No. 9, "Debris-Control Structures", shows types of debris control structures and provides a guide for selecting the type of structure suitable for various debris classifications.

Topic 823 - Culvert Location

823.1 Introduction

The culvert usually should be located so that the thalweg of the stream to be accommodated, approaches and exits at the approximate centerline of the culvert. However, for economic reasons, as a general rule, small skews should be eliminated, moderate skews retained and large skews reduced.

Since the culvert typically acts as a constriction, local velocities will increase through the barrel and in the vicinity of the outlet. The location and design must be also sensitive to the environment (fish passage etc).

As a general rule, flood waters should be conducted under the highway at first opportunity minimizing scour of embankment and entrapment of debris. Therefore, culverts should be placed at each defined swale to limit carryover of drainage from one watershed to another.

823.2 Alignment and Slope

The ideal culvert placement is on straight alignment and constant slope. Variations from a straight alignment should be only to accommodate unusual conditions. Where conditions require deviations from the tangent alignment, abrupt changes in direction or slope should be avoided in order to maintain the hydraulic efficiency, and avoid excessive maintenance. Angle points may be permissible in the absence of abrasives in the flow; otherwise, curves should be used. When angle points are unavoidable, maintenance access may be necessary. See Index 838.5 for manhole location criteria.

Curvature in pipe culverts is obtained by a series of angle points. Whenever conditions require these angle points in culvert barrels, the number of angle points must be specified either in the plans or in the special provisions. The angle can vary depending upon conditions at the site, hydraulic requirements, and purpose of the culvert. The angle point requirement is particularly pertinent if there is a likelihood that structural steel plate pipe will be used. The structural steel plate pipe fabricator must know what the required miters are in order for the plates to be fabricated satisfactorily. Manufacturers' literature should be consulted to be sure that what is being specified can be fabricated without excessive cost.

Ordinarily the grade line should coincide with the existing streambed. Deviations from this practice are permissible under the following conditions:

- (a) On flat grades where sedimentation may occur, place the culvert inlet and outlet above the streambed but on the same slope. The distance above the streambed depends on the size length and amount of sediment anticipated.

If possible, a slope should be used that is sufficient to develop self-cleaning velocities.

- (b) Under high fills, anticipate greater settlement under the center than the sides of the fill. Where settlement is anticipated, provisions should be made for camber.

- (c) In steep sloping areas such as on hillsides, the overfill heights can be reduced by designing the culvert on a slope flatter than natural slope. However, a slope should be used to maintain a velocity sufficient to carry the bedload. A spillway or downdrain can be provided at the outlet. Outlet protection should be provided to prevent undermining. For the downdrain type of installation, consideration must be given to anchorage. This design is appropriate only where substantial savings will be realized.

Topic 824 - Culvert Type Selection

824.1 Introduction

A culvert is a hydraulically short conduit which conveys stream flow through a roadway embankment or past some other type of flow obstruction. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert selection factors include roadway profiles, channel characteristics, flood damage evaluations, construction and maintenance costs, and estimates of service life.

824.2 Shape and Cross Section

- (1) Numerous cross-sectional shapes are available. The most commonly used shapes include circular, box (rectangular), elliptical, pipe-arch, and arch. The shape selection is based on the cost of construction, the limitation on upstream water surface elevation, roadway embankment height, and hydraulic performance.
- (2) *Multiple Barrels.* In general, the spacing of pipes in a multiple installation, measured between outside surfaces, should be at least half the nominal diameter with a minimum of 2 feet.

See Standard Plan D89 for multiple pipe headwall details.

Additional clearance between pipes is required to accommodate flared end sections. See

Standard Plans, D94A & B for width of flared end sections.

Topic 825 - Hydraulic Design of Culverts

825.1 Introduction

After the design discharge, (Q), has been estimated, the conveyance of this water must be investigated. This aspect is referred to as hydraulic design.

The highway culvert is a special type of hydraulic structure. An exact theoretical analysis of culvert flow is extremely complex because the flow is usually non-uniform with regions of both gradually varying and rapidly varying flow. Hydraulic jumps often form inside or downstream of the culvert barrel. As the flow rate and tailwater elevations change, the flow type within the barrel changes. An exact hydraulic analysis therefore involves backwater and drawdown calculations, energy and momentum balance, and application of the results of hydraulic studies.

An extensive hydraulic analysis is usually impractical and not warranted for the design of most highway culverts. The culvert design procedures presented herein and in the referenced publications are accurate, in terms of head, to within plus or minus 10 percent.

825.2 Culvert Flow

The types of flow and control used in the design of highway culverts are:

- Inlet Control - Most culverts operate under inlet control which occurs when the culvert barrel is capable of carrying more flow than the inlet will accept. Supercritical flow is usually encountered within the culvert barrel. When the outlet is submerged under inlet control, a hydraulic jump will occur within the barrel.
- Outlet Control - Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet will accept. Culverts under outlet control generally function with submerged outlets

and subcritical flow within the culvert barrel. However, it is possible for the culvert to function with an unsubmerged outlet under outlet control where flow passes through critical depth in the vicinity of the outlet.

For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross sectional area of the culvert, inlet geometry, and elevation of headwater at entrance are of primary importance. Outlet control involves the additional consideration of the tailwater elevation of the outlet channel and the slope, roughness and length of the culvert barrel. A discussion of these two types of control with charts for selecting a culvert size for a given set of conditions is included in the FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts."

825.3 Computer Programs

Numerous calculator and computer programs are available to aid in the design and analysis of highway culverts. The major advantages of these programs over the traditional hand calculation method are:

- Increased accuracy over charts and nomographs.
- Rapid comparison of alternative sizes and inlet configurations.

Familiarity with culvert hydraulics and traditional methods of solution is necessary to provide a solid basis for designers to take advantage of the speed, accuracy, and increased capabilities of hydraulic design computer programs.

The hydraulic design calculator and computer programs available from the FHWA are more fully described in HDS No. 5, "Hydraulic Design of Highway Culverts."

The HY8 culvert hydraulics program provides interactive culvert analysis. Given all of the appropriate data, the program will compute the culvert hydraulics for circular, rectangular, elliptical, arch, and user-defined culverts.

The logic of HY8 involves calculating the inlet and outlet control headwater elevations for the given

flow. The elevations are then compared and the larger of the two is used as the controlling elevation. In cases where the headwater elevation is greater than the top elevation of the roadway embankment, an overtopping analysis is done in which flow is balanced between the culvert discharge and the surcharge over the roadway. In the cases where the culvert is not full for any part of its length, open channel computations are performed.

825.4 Coefficient of Roughness

Suggested Manning's n values for culvert design are given in Table 852.1.

Topic 826 - Entrance Design

826.1 Introduction

The size and shape of the entrance are among the factors that control the level of ponding at the entrance. Devices such as rounded or beveled lips and expanded entrances help maintain the velocity of approach, increase the culvert capacity, and may lower costs by permitting a smaller sized culvert to be used.

The inherent characteristics of common entrance treatments are discussed in Index 826.4. End treatment on large culverts is an important consideration. Selecting an appropriate end treatment for a specific type of culvert and location requires the application of sound engineering judgment.

The FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts" combines culvert design information previously contained in HEC No. 5, No. 10, and No. 13. The hydraulic performance of various entrance types is described in HDS No. 5.

826.2 End Treatment Policy

The recommended end treatment for small culverts is the prefabricated flared end section. For safety, aesthetic, and economic reasons, flared end sections should be used at both entrance and outlet whenever feasible instead of headwalls.

End treatment, either flared end section or headwall, is required for circular culverts 60 inches or more in diameter and for pipe arches of equivalent size.

826.3 Conventional Entrance Designs

The inlet edge configuration is one of the prime factors influencing the hydraulic performance of a culvert operating in inlet control. The following entrance types are frequently used.

(1) *Projecting Barrel.* A thin edge projecting inlet can cause a severe contraction of the flow. The effective cross sectional area of the barrel may be reduced to about one half the actual available barrel area.

The projecting barrel has no end treatment and is the least desirable hydraulically. It is economical but its appearance is not pleasing and use should be limited to culverts with low velocity flows where head conservation, traffic safety, and appearance are not important considerations.

Typical installations include an equalizer culvert where ponding beyond the control of the highway facility occurs on both sides of the highway or where the flow is too small to fill the minimum culvert opening.

The projecting entrance inhibits culvert efficiency. In some situations, the outlet end may project beyond the fill, thus providing security against erosion at less expense than bank protection work.

Projecting ends may prove a maintenance nuisance, particularly when clearance to right of way fence is limited.

(2) *Flared End Sections.* This end treatment provides approximately the same hydraulic performance as a square-edge headwall and is used to retain the embankment, improve the aesthetics, and enhance safety. Because prefabricated flared end sections provide better traffic safety features and are considered more attractive than headwalls they are to be used instead of headwalls whenever feasible.

Details of prefabricated flared end sections for circular pipe in sizes 12 inches through 84 inches in diameter and pipe arches of equivalent size are shown on Standard Plans D94A & B.

(3) *Headwalls and Wingwalls.* This end treatment may be required at the culvert entrance for the following reasons:

- To improve hydraulic efficiency.
- To retain the embankment and reduce erosion of slopes.
- To provide structural stability to the culvert ends and serve as a counterweight to offset buoyant or uplift forces.

(4) *Rounded Lip.* This treatment costs little, smoothes flow contraction, increases culvert capacity, and reduces the level of ponding at the entrance. The box culvert and pipe headwall standard plans include a rounded lip. The rounded lip is omitted for culverts less than 48 inches in diameter; however, the beveled groove end of concrete pipe at the entrance produces an effect similar to that of a rounded lip.

(5) *Mitered End.* A mitered culvert end is formed when the culvert barrel is cut to conform with the plane of the embankment slope. Mitered entrances are not to be used. They are hydraulically less efficient than either flared end sections or headwalls, and they are structurally unstable.

(6) *Entrance Risers.* At a location where the culvert would be subject to plugging, a vertical pipe riser should be considered. Refer to Index 822.2 for discussion on debris-control structures.

826.4 Improved Inlet Designs

Entrance geometry refinements can be used to reduce the flow contraction at the inlet and increase the capacity of culverts operating under inlet control without increasing the headwater depth. The following entrance types improve culvert inlet performance and can be provided at reasonable cost.

(1) *Expanded Entrances.* Headwalls with straight flared wingwalls or warped wingwalls offer a more highly developed entrance appropriate for large culverts, regardless of type or shape of barrel. The effect of such entrances can be approximated more economically by a shaped entrance using air blown mortar, concreted riprap, sacked concrete or slope paving.

Straight flared wingwalls and warped wingwalls aid in maintaining the approach velocity, align and guide drift, and funnel the flow into the culvert entrance. To insure enough velocity to carry drift and debris through the culvert or increase the velocity and thereby increase the entrance capacity, a sloping drop down apron at the entrance may be used. To minimize snagging drift, the standard plans require wingwalls to be flush with the culvert barrel. The flare angle may range from 30 to 75 degrees; the exact angle is based on the alignment of the approach channel banks and not the axis of the culvert. Greater efficiency is obtained when the top of the wingwall is the same elevation as the headwall.

Whether warped or straight flared wingwalls are used depends on the shape of the approach channel. Straight flared wingwalls are appropriate for well defined channels with steep banks. Warped wingwalls are more suited to shallow trapezoidal approach channels.

Usually it is more economical to transition between the stream section and the culvert by means of straight flared wingwalls or warped wingwalls than to expand the culvert barrel at entrance. For a very wide channel, this transition may be combined with riprap, dikes, or channel lining extending upstream to complete the transition.

(2) *Transitions.* Elaborate transitions and throated openings for culverts may be warranted in special cases. Generally a highly developed entrance is unnecessary if the shape of the culvert fits the approach channel. In wide flat channels where ponding at entrance must be restricted, a wide shallow structure or multiple

conduit should be used if drift and debris are not a problem.

Throated or tapered barrels at entrance are more vulnerable to clogging by debris. They are not economical unless they are used for corrective measures; for example, where there is a severe restriction in right of way width and it is necessary to increase the capacity of an existing culvert structure.

For further information refer to HEC-9, "Debris-Control Structures" and HDS 5, "Hydraulic Design of Highway Culverts"

Topic 827 - Outlet Design

827.1 General

The outlet velocity of highway culverts is usually higher than the maximum natural stream velocity. This higher velocity can cause streambed scour and bank erosion for a limited distance downstream from the culvert outlet.

The slope and roughness of the culvert barrel are the principle factors affecting outlet velocity. The shape and size of a culvert seldom have a significant effect on the outlet velocity. When the outlet velocity is believed to be excessive and it cannot be satisfactorily reduced by adjusting the slope or barrel roughness, it may be necessary to use some type of outlet protection or energy dissipator. A method of predicting and analyzing scour conditions is given in the FHWA publication "Scour at Culvert Outlets in Mixed Bed Materials", FHWA/RD - 82/011.

When dealing with erosive velocities at the outlet, the effect on downstream property should be evaluated.

827.2 Embankment Protection

Improved culvert outlets are designed to restore natural flow conditions downstream. Where erosion is to be expected, corrective measures such as bank protection, vertical flared wingwalls, warped wingwalls, transitions, and energy dissipators may be considered. See Chapter 870, "Channel and Shore Protection-Erosion Control", FHWA Hydraulic Engineering Circulars No. 11,

"Design of Riprap Revetment", No. 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels", and No. 15, "Design of Roadway Channels with Flexible Linings", and "Hydraulic Design of Stilling Basins and Energy Dissipators", Engineering Monograph No. 25 by the U. S. Department of Interior, Bureau of Reclamation, 1964 (revised 1978). HY-8, within the Hydrain Integrated Computer Program System, provides designs for energy dissipators and follows the HEC-14 method for design.

Culvert outlet design should provide a transition for the 100-year flood or design event from the culvert outlet to a section in the natural channel where natural stage, width, and velocity will be restored, or nearly so, with consideration of stability and security of the natural channel bed and banks against scour.

If an outfall structure is required for transition, typically it will not have the same design as the entrance.

Wingwalls, if intended for an outlet transition (expansion), generally should not flare at an angle (in degrees) greater than 150 divided by the outlet velocity in feet per second. However, transition designs fall into two general categories: those applicable to culverts in outlet control (subcritical flow) or those applicable to culverts in inlet control (supercritical). The procedure outlined in HEC-14 for subcritical flow expansion design should also be used for supercritical flow expansion design if the culvert exit Froude number (Fr) is less than 3, if the location where the flow conditions desired is within 3 culvert diameters of the outlet, and if the slope is less than 10 percent. For supercritical flow expansions outside these limits, the energy equation can be used to determine flow conditions leaving the transition.

Warped endwalls can be designed to fit trapezoidal or U-shaped channels, as transitions for moderate-to-high velocity (10 feet per second – 18 feet per second).

For extreme velocity (exceeding 18 feet per second) the transition can be shortened by using an energy-dissipating structure.

Topic 828 - Diameter and Length

828.1 Introduction

From a maintenance point of view the minimum diameter of pipe and the distance between convenient cleanout access points are important considerations.

The following instructions apply to minimum pipe diameter and the length of pipe culvert.

828.2 Minimum Diameter

The minimum diameter for cross culverts under the roadway is 18 inches. For other than cross pipes, the minimum diameter is 12 inches. For maintenance purposes, where the slope of longitudinal side drains is not sufficient to produce self-cleaning velocities, pipe sizes of 18 inches or more in diameter should be considered.

The minimum diameter of pipe to be used is further determined by the length of pipe between convenient cleanout access points. If pipe runs exceed 100 feet between inlet and outlet, or intermediate cleanout access, the minimum diameter of pipe to be used is 24 inches. When practicable, intermediate cleanout points should be provided for runs of pipe 24 inches in diameter that exceed 300 feet in length.

If a choice is to be made between using 18-inch diameter pipe with an intermediate cleanout in the highway median or using 24-inch diameter pipe without the median access, the larger diameter pipe without the median access is preferred.

828.3 Length

The length of pipe culvert to be installed is determined as follows:

- (a) Establish a theoretical length based on slope stake requirements making allowance for end treatment.
- (b) Adjust the theoretical length for height of fill by applying these rules:
 - For fills 12 feet or less, no adjustment is required.

- For fills higher than 12 feet, add 1 foot of length at each end for each 10 foot increment of fill height or portion thereof. The additional length should not exceed 6 feet on each end.
 - In cases of high fills with benches, the additional length is based on the height of the lowest bench.
- (c) Use the nearest combination of commercial lengths which equal or exceed the length obtained in (b) above.

Topic 829 - Special Considerations

829.1 Introduction

In addition to the hydraulic design, other factors must be considered to assure the integrity of culvert installations and the highway.

829.2 Bedding and Backfill

The height of overfill a culvert will safely sustain depends upon foundation conditions, method of installation, and its structural strength and rigidity.

Uniform settlement under both the culvert and the adjoining fill will not overstress flexible and segmental rigid culverts. Unequal settlement, however, can result in distortion and shearing action in the culvert. For rigid pipes this could result in distress and disjuncting of the pipe. A flexible culvert accommodates itself to moderate unequal settlements but is also subject to shearing action. Monolithic culverts can tolerate only a minimal amount of unequal settlement, and require favorable foundation conditions. Any unequal settlement would subject a monolithic culvert to severe shear stresses.

- (1) *Foundation Conditions.* A slightly yielding foundation under both the culvert and adjoining fill is the foundation condition generally encountered. The maximum height of cover tables given in Chapter 850 are based on this foundation condition.

Unyielding foundation conditions can produce high stresses in the culverts. Such stresses may be counteracted by subexcavation and backfill.

The Standard Plans show details for shaped, sand, and soil cement bedding treatments.

Foundation materials capable of supporting pressures between 1.0 tons per square foot and 8.0 tons per square foot are required for culverts with cast-in-place footing or inverts, such as reinforced concrete boxes, arches, and structural plate arches. When culvert footing pressures exceed 1.5 tons per square foot or the diameter or span exceeds 10 feet, a geology report providing a log of test boring is required.

Adverse foundation and backfill conditions may require a specially designed structure. The allowable overfill heights for concrete arches, structural plate arches, and structural plate vehicular undercrossings are based on existing soil withstanding the soil pressures indicated on the Standard Plans. A foundation investigation should be made to insure that the supporting soils withstand the design soil pressures for those types of structures.

- (2) *Method of Installation.* Under ordinary conditions, the methods of installation described in the Standard Specifications and shown on the Standard Plans should be used. For any predictable settlement, provisions for camber should be made.

Excavation and backfill details for circular concrete pipe, reinforced box and arch culverts, and corrugated metal pipe and arch culverts are shown on Standard Plans A62-D, A62DA, A62-E, and A62-F respectively.

- (3) *Height of Cover.* There are several alternative materials from which acceptable culverts may be made. Tables of maximum height of cover recommended for the more frequently used culvert shapes, sizes, corrugation configurations, and types of materials are given in Chapter 850. Not included, but covered in the Standard Plans, are maximum earth cover for reinforced concrete box culverts, reinforced concrete arches, and structural plate vehicular undercrossing.

For culverts where overfill requirements exceed the limits shown on the tables a special design must be prepared. Special designs are to be submitted to the Division of Structures for review, or the Division of Structures may be directly requested to prepare the design.

Under any of the following conditions, the Division of Structures is to prepare the special design:

- Where foundation material will not support footing pressure shown on the Standard Plans for concrete arch and structural plate vehicular undercrossings.
- Where foundation material will not support footing pressures shown in the Highway Design Manual for structural plate pipe arches or corrugated metal pipe arches.
- Where a culvert will be subjected to unequal lateral pressures, such as at the toe of a fill or adjacent to a retaining wall.

Special designs usually require that a detailed foundation investigation be made.

- (4) *Minimum Cover.* When feasible, culverts should be buried at least 1 foot. For construction purposes, a minimum cover of 6 inches greater than the thickness of the structural cross section is desirable for all types of pipe. The minimum thickness of cover for various type culverts under rigid or flexible pavements is given in Table 856.5.

829.3 Piping

Piping is a phenomenon caused by seepage along a culvert barrel which removes fill material, forming a hollow similar to a pipe. Fine soil particles are washed out freely along the hollow and the erosion inside the fill may ultimately cause failure of the culvert or the embankment.

The possibility of piping can be reduced by decreasing the velocity of the seepage flow. This can be reduced by providing for watertight joints. Therefore, if piping through joints could become a problem, consideration should be given to providing for watertight joints.

Piping may be anticipated along the entire length of the culvert when ponding above the culvert is expected for an extended length of time, such as when the highway fill is used as a detention dam or to form a reservoir. Headwalls, impervious materials at the upstream end of the culvert, and anti-seep or cutoff collars increase the length of the flow path, decrease the hydraulic gradient and the velocity of flow and thus decrease the probability of piping developing. Anti-seep collars usually consist of bulkhead type plate or blocks around the entire perimeter of the culvert. They may be of metal or concrete, and, if practical, should be keyed into impervious material.

Piping could occur where a culvert must be placed in a live stream, and the flow cannot be diverted. Under these conditions watertight joints should be specified.

829.4 Joints

The possibility of piping being caused by open joints in the culvert barrel may be reduced through special attention to the type of pipe joint specified. For a more complete discussion of pipe joint requirements see Index 854.1.

The two pipe joint types specified for culvert installations are identified as "standard" and "positive". The "standard" joint is adequate for ordinary installations and "positive" joints should be specified where there is a need to withstand soil movements or resist disjoining forces. Corrugated metal pipe coupling band details are shown on Standard Plan sheets D97A through D97G and concrete pipe joint details on sheet D97H.

If it is necessary for "standard" or "positive" joints to be watertight they must be specifically specified as such. Rubber "O" rings or other resilient joint material provides the watertight seal. Corrugated metal pipe joints identified as "downrain" are watertight joint systems with a tensile strength specification for the coupler.

829.5 Anchorage

Refer to Index 834.4(5) for discussion on anchorage for overside drains.

Reinforced concrete pipe should be anchored and have positive joints specified if either of the following conditions is present:

- (a) Where the pipe diameter is 60 inches or less, the pipe slope is 33 percent or greater, and the fill over the top of the pipe less than 1.5 times the outside diameter of the pipe measured perpendicular to the slope.
- (b) Where the pipe diameter is greater than 60 inches and the pipe slope is 33 percent or greater, regardless of the fill over the top of the pipe.

Where the slopes have been determined by the geotechnical engineer to be potentially unstable, regardless of the slope of the pipe, as a minimum, the pipes shall have positive joints. Alternative pipes/anchorage systems shall be investigated when there is a potential for substantial movement of the soil.

Where anchorage is required, there should be a minimum of 18 inches cover measured perpendicular to the slope.

Typically buried flexible pipe with corrugations on the exterior surface will not require anchorage, however, a special detail will be required for plastic pipe without corrugations on the exterior surface.

829.6 Irregular Treatment

- (1) *Junctions.* (Text Later)
- (2) *Bends.* (Text Later)

829.7 Siphons and Sag Culverts

- (1) *General Notes.* There are two kinds of conduits called siphons: the true siphon and the inverted siphon or sag culvert. The true siphon is a closed conduit, a portion of which lies above the hydraulic grade line. This results in less than atmospheric pressure in that portion. The sag culvert lies entirely below the hydraulic grade line; it operates under pressure without siphonic action.

Under the proper conditions, there are hydraulic and economic advantages to be

obtained by using the siphon principle in culvert design.

(2) *Sag Culverts*. This type is most often used to carry an irrigation canal under a highway when the available headroom is insufficient for a normal culvert. The top of a sag culvert should be at least 4.5 feet below the finished grade where possible, to ensure against damage from heavy construction equipment. The culvert should be on a straight grade and sumps provided at each end to facilitate maintenance. Sag culverts should not be used:

- (a) When the flow carries trash and debris in sufficient quantity to cause heavy deposits,
- (b) For intermittent flows where the effects of standing water are objectionable, or
- (c) When any other alternative is possible at reasonable cost.

(3) *Types of Conduit*. Following are two kinds of pipes used for siphons and sag culverts to prevent leakage:

- (a) Reinforced Concrete Pipe - Reinforced concrete pipe with joint seals is generally satisfactory. For heads over 20 feet, special consideration should be given to hydrostatic pressure.
- (b) Corrugated Metal Pipe - corrugated metal pipe must be of the thickness and have the protective coatings required to provide the design service life. Field joints must be watertight. The following additional treatment is recommended.

- When the head is more than 10 feet and the flow is continuous or is intermittent and of long duration, pipe fabricated by riveting, spot welding or continuous helical lockseam should be soldered.

Pipe fabricated by a continuous helical welded seam need not be soldered.

- If the head is 10 feet or less and the flow is intermittent and lasts only a few days, as in storm flows, unsoldered seams are permissible.

829.8 – Currently Not In Use

829.9 Dams

Typically, proposed construction which is capable of impounding water to the extent that it meets the legal definition of a dam must be approved by the Department of Water Resource (DWR), Division of Safety of Dams. The legal definition is described in Sections 6002 and 6003 of the State Water Code. Generally, any facility 25 feet or more in height or capable of impounding 50 acre-feet or more would be considered a dam. However, any facility 6 feet or less in height, regardless of capacity, or with a storage capacity of not more than 15 acre-feet, regardless of height, shall not be considered a dam. Additionally, Section 6004 of the State Water Code states "... and no road or highway fill or structure ... shall be considered a dam." Therefore, except for large retention or detention facilities there will rarely be the need for involvement by the DWR in approval of Caltrans designs.

Although most highway designs will be exempt from DWR approval, caution should always be exercised in the design of high fills that could impound large volumes of water. Even partial plugging of the cross drain could lead to high pressures on the upstream side of the fill, creating seepage through the fill and/or increased potential for piping.

The requirements for submitting information to the FHWA Division Office in Sacramento as described in Index 805.6 are not affected by the regulations mentioned above.

829.10 Reinforced Concrete Box Modifications

(1) *Extensions*. Where an existing box culvert is to be lengthened, it is essential to perform an on-site investigation to verify the structural integrity of the box. If signs of distress are present, the Division of Structures must be contacted prior to proceeding with the design.

(2) *Additional Loading*. When significant additional loading is proposed to be added to an existing reinforced concrete box culvert the Division of Structures must be contacted prior

to proceeding with the design. Overlays of less than 6 inches in depth, or widenings that do not increase the per unit loading on the box are not considered to be significant. Designers should also check the extent that previous projects might have increased loading on box culverts, even if the current project is not adding a significant amount of loading.

- Limit pond duration and depth (see Topic 833)
- (4) Overtopping
- Avoid overtopping at cross culverts using appropriate freeboard and/or headwater elevation (see Topic 821)

Where suitable measures cannot be implemented to address conditions such as those identified above, or an identified existing problem area, coordination should be made with the Safety Review Committee per Index 110.8.

831.5 Computer Programs

There are many computer programs available to aid highway design engineers with estimating runoff and ensuing hydraulic design and analysis of roadway drainage facilities.

Refer to Table 808.1 for guidance on selecting appropriate software programs for specific analysis needs.

Familiarity with the fundamentals of hydraulics and traditional methods of solution are necessary to assure that the results obtained are reasonable. There is a tendency for inexperienced engineers to accept computer output as valid without verifying the reasonableness of input and output data.

Topic 832 - Hydrology

832.1 Introduction

The philosophy and principles of hydrology are discussed in Chapter 810. Additional information on methods of estimating storm runoff may be found in FHWA's HEC 22.

832.2 Rational Method

With few exceptions, runoff estimates for roadway drainage design are made by using Rational Methods described under Index 819.2(1). In order to make use of these methods, information on the intensity, duration, and frequency of rainfall for the locality of the project must be established. Refer to Index 815.3(3) for further information on precipitation intensity-duration-frequency (IDF) curves that have been developed for many locations in California.

832.3 Time of Concentration

Refer to Index 816.6 for information on time of concentration.

Topic 833 - Roadway Cross Sections

833.1 Introduction

The geometric cross section of the roadway affects drainage features and hydraulic considerations. Cross slope and width of pavement and shoulders as well as other roadway geometry affect the rate of runoff, width of tolerable spread, and hydraulic design considerations. The cross section of drainage features such as, depressed medians, curbs and gutters, dikes, and side ditches is often controlled by an existing roadway geometric cross section or the one selected for new highway construction.

833.2 Grade, Cross Slope and Superelevation

The longitudinal slope or grade is governed by the highway grade line as discussed under Index 831.2. Refer to Index 204.3 for minimum grade and Indexes 301.3 and 302.2 for cross slope. Where three (3) lanes or more are sloped in the same direction, it is desirable to counter the resulting increase in flow depth by increasing the cross slope of the outermost lanes. The two (2) lanes adjacent to the crown line should be pitched at the normal slope, and successive lane pairs, or portions thereof outward, should be increased by about 0.5 to 1 percent. The maximum pavement cross slope should be limited to 4 percent. However, exceptions to the design criteria for cross slope in Index 302.2 must be formally approved in accordance with the requirements Index 82.2, "Approvals for Nonstandard Design." For projects where lanes will be added on the inside of divided highways, or when widening an existing "crowned" 2-lane highway to a 4-lane divided highway, consideration should be given to the use of a "tent section" in order to minimize the number of lanes sloping in the same direction. Refer to Index 301.2. Consideration should be given to increasing cross slopes in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades. Superelevation is discussed in Topic 202. Refer to Index 831.4 for Hydroplaning considerations.

Topic 834 - Roadside Drainage

834.1 General

Median drainage, ditches and gutters, and overside drains are some of the major roadside drainage facilities.

834.2 Median Drainage

(1) *Drainage Across the Median.* When it is necessary for sheet flow to cross flush medians, it should be intercepted by the use of slotted drains or other suitable alternative facilities. See Standard Plan D98-B for slotted drain details.

Where floodwaters are allowed to cross medians, designers must consider the impacts of railings, barrier or other obstructions to both the depth and spread of flow. Designers should consult their district hydraulic unit for assistance.

(2) *Grade and Cross Slope.* The longitudinal slope or grade for median drainage is governed by the highway grade line as discussed under Index 831.2. Refer to Index 204.3 for minimum grade and Indexes 305.2 and 405.5(4) for standards governing allowable cross slope of medians.

Existing conditions control median grades and attainable cross slope on rehabilitation projects. The flattest desirable grade for earth medians is 0.25 percent and 0.12 percent for paved gutters in the median.

(3) *Erosion.* When velocities are excessive for soil conditions, provisions for erosion control should be provided. See Table 865.2 for recommended permissible velocities for unlined channels.

Economics and aesthetics are to be taken into consideration in the selection of median erosion control measures. Under the less severe conditions, ground covers of natural or synthetic materials which render the soil surface stable against accelerated erosion are adequate. Under the more severe conditions, asphalt or concrete ditch paving may be required.

Whenever median ditch paving is necessary, consideration should be given to the use of

cement or lime treatment of the soil. The width treated will depend on the capacity needed to handle the drainage. A depth of 6 inches is generally satisfactory. The amount of cement or lime to be used should be based on laboratory tests of the in-place material to be tested, and normally varies from 6 percent to 10 percent. If a clear or translucent curing compound is used, the completed area is unobtrusive and aesthetically pleasing.

Asphalt concrete ditch paving and soil cement treatments cured with an application of liquid asphalt are highly visible and tend to become unsightly from streaks of eroded material. Cobbles, though effective for erosion control, are not satisfactory in a recovery area for out of control vehicles. See Topic 872 for further discussion on erosion protection and additional types of ditch linings. Erosion control references are given under Index 871.3.

(4) *Economy in Design.* Economy in median drainage can be achieved by locating inlets to utilize available nearby culverts or the collector system of a roadway drainage installation. The inlet capacity can be increased by placing it in a local depression. Use of slotted pipe at sag points where a local depression might be necessary may be an alternative solution to a grate catch basin.

834.3 Ditches and Gutters

(1) *Grade.* The flattest grade recommended for design is 0.25 percent for earth ditches and 0.12 percent for paved ditches.

(2) *Slope Ditches.* Slope ditches, sometimes called surface, brow, interception, or slope protection ditches, should be provided at the tops of cuts where it is necessary to intercept drainage from natural slopes inclined toward the highway.

When the grade of a slope ditch is steep enough that erosion would occur, the ditch should be paved. Refer to Table 865.2 for permissible velocities for unlined channels in various types of soil. When the ditch grade exceeds a 4:1 slope, a downdrain is advisable. Slope ditches may not be necessary where side slopes in favorable soils are flatter than 2:1 or where positive erosion control measures are to be instituted during construction.

838.5 Appurtenant Structures

(1) Manholes.

- (a) General Notes. The purpose of a manhole is to provide access to a storm drain for inspection and maintenance. Manholes are usually constructed out of cast in place concrete, pre-cast concrete, or corrugated metal pipe. They are usually circular and approximately three or four feet in diameter to facilitate the movement of maintenance personnel.

There is no Caltrans Standard Plan for manholes. Relocation and reconstruction of existing storm drain facilities, owned by a city or county agency, is often necessary. Generally the local agency has adopted manhole design standard for use on their facilities. Use of the manhole design preferred by the responsible authority or owner is appropriate.

Commercial precast manhole shafts are effective and usually more economical than cast in place shafts. Brick or block may also be used, but only upon request and justification from the local agency or owner.

- (b) Location. Following are common locations for manholes:
- Where two or more drains join,
 - At locations and spacing which facilitate maintenance,
 - Where the drain changes in size,
 - At sharp curves or angle points in excess of 10 degrees,
 - Points where an abrupt flattening of the grade occurs, and
 - On the smaller drains, at the downstream end of a sharp curve.

Manholes are not required if the conduit is large enough to accommodate a man, unless spacing criteria govern. Manholes should not be placed within the traveled way. Exceptions are frontage roads and city streets, but intersection locations should be avoided.

- (c) Spacing. In general, the larger the storm drain, the greater the manhole spacing. For pipe diameter of 48 inches or more, or other shapes of equal cross sectional area, the manhole spacing ranges from 700 feet to 1200 feet. For diameters of less than 48 inches, the spacing may vary from 300 feet to 700 feet. In the case of small drains where self-cleaning velocities are unobtainable, the 300 feet spacing should be used. With self-cleaning velocities and alignments without sharp curves, the distance between manholes should be in the upper range of the above limits.
- (d) Access Shaft. For drains less than 48 inches in diameter, the access shaft is to be centered over the drain. When the drain diameter exceeds the shaft diameter, the shaft should be offset and made tangent to one side of the pipe for better location of the manhole steps. For drains 48 inches or more in diameter, where laterals enter from both sides of the manhole, the offset should be toward the side of the smaller lateral. See Standard Plan D93A for riser connection details.
- (e) Arrangement of Laterals. To avoid unnecessary head losses, the flow from laterals which discharge opposite each other should converge at an angle in the direction of flow. If conservation of head is critical, a training wall should be provided.
- (2) *Junction Structures.* A junction structure is an underground chamber used to join two or more conduits, but does not provide direct access from the surface. It is designed to prevent turbulence in the flow by providing a smooth transition. This type of structure is usually needed only where the trunk drain is 42 inches or more in diameter. A standard detailsheet of a junction structure is available for pipes ranging from 42 inches to 84 inches in diameter at the following Office Engineer web site address: http://www.dot.ca.gov/hq/esc/structures_cadd/XS_sheets/Metric/dgn/. The XS sheet reference is XS 4-26. Where required by spacing criteria, a manhole should be used.

(3) *Flap Drainage gates.* When necessary, backflow protection should be provided in the form of flap drainage gates. These gates offer negligible resistance to the release of water from the system and their effect upon the hydraulics of the system may be neglected.

If the outlet is subject to floating debris, a shelter should be provided to prevent the debris from clogging the flap drainage gate. Where the failure of a flap drainage gate to close would cause serious damage, a manually controlled gate in series should be considered for emergencies.

Topic 839 - Pumping Stations

839.1 General

Drainage disposal by pumping should be avoided where gravity drainage is reasonable. Because pumping installations have high initial cost, maintenance expense, power costs, and the possibility of failure during a storm, large expenditures can be justified for gravity drainage. In some cases, this can be accomplished with long runs of pipe or continuing the depressed grade to a natural low area.

Whenever possible, drainage originating outside the depressed areas should be excluded. District and Division of Structures cooperation is essential in the design of pumping stations, tributary storm drains, and outfall facilities. This is particularly true of submerged outlets, outlets operating under pressure, and outlets of unusual length.

839.2 Pump Type

Horizontal pumps in a dry location are generally specified for ease of access, safety, and standardization of replacement parts.

Only in special cases is stand-by power for pumping plants a viable consideration. All proposals for stand-by power are to be reviewed by and coordinated with the Division of Structures.

839.3 Design Responsibilities

When a pumping station is required, responsibility for design between the District and the Division of Structures is as follows:

(1) *Districts.* The District designs the collector and the outfall facilities leading from the chamber into which the pumps discharge. This applies to outfalls operating under gravity and with a free outlet. Refer to Topic 838.

Details of pumping stations supportive information to be submitted by the District to the Division of Structures is covered under Index 805.8 and Chapter 3-3.1(4) of the Drafting and Plans Manual.

(2) *Division of Structures.* The Division of Structures will prepare the design and contract plans for the pumping station, the storage box and appurtenant equipment, considering the data and recommendations submitted by the District.

The Division of Structures will furnish the District a preliminary plan based on data previously submitted by the District. It will show the work to be covered by the Division of Structures plans, including a specific location for the pumping plant and storage box, the average and maximum pumping rates and the power required.

839.4 Trash and Debris Considerations

Storm drain systems leading to pumping plants are to be designed to limit the inflow of trash and debris, as these may cause damage to the pump impellers and create a maintenance removal nuisance. Standard grate designs are effective at ensuring that trash and debris are screened out of the inflow, but where side opening or curb opening inlets are constructed, trash racks must be added to the inlet design. The only Standard Plan detail for curb opening designs is shown on Standard Plan D74B and is used in conjunction with Type GDO inlets. On those occasions where pipe risers with side opening inlets are part of the system, refer to Standard Plan D93C for appropriate trash rack design details.

839.5 Maintenance Consideration

Access to the pumping plant location for both maintenance personnel and maintenance vehicles is generally provided by way of paved access road or city street. One parking space minimum is to be provided in the vicinity of the pumping plant. An area light is generally provided when it is

determined that neither the highway lighting nor the street lighting is adequate. Access to the pumping plant for maintenance from the top of the cut slope generally consists of a stairway located adjacent to the pumping plant. The stairway generally extends from the top of cut slope to the toe of cut slope. Access to the pump control room should be through a vertical doorway with the bottom above flood level, and never through a hatch.

839.6 Groundwater Considerations

As the lowest point in the storm drain system, pumping plants are particularly susceptible to problems associated with rises in groundwater tables. Where the foundation of pump houses or associated storage boxes are at an elevation where they would be subjected to existing or future groundwater tables, sealing around the base of the foundation is necessary. The use of bentonite or other impervious material is typically sufficient in keeping groundwater from welling up through the relatively pervious structure backfill.

Sealing requirements will typically be specified by the Division of Structures during the pump plant design. However, the district should provide any information relative to historical groundwater levels or fluctuations which would be of importance, or known plans by local or regional water districts to modify recharge patterns in a manner that could impact the design.

CHAPTER 900 LANDSCAPE ARCHITECTURE

Topic 901 - General

Index 901.1 - Landscape Architecture Program

The Landscape Architecture Program is responsible for the development of policies, programs, procedures, and standards for all aspects of the Roadside Program which consists of highway planting, replacement highway planting, mitigation planting, highway planting revegetation, highway planting restoration, roadside rehabilitation, roadside protection and restoration, roadside improvements, safety roadside rest areas, scenic highways, classified landscaped freeways, transportation art, gateway monuments, community identification, blue star memorial highways, and planting in conjunction with noise barriers.

This chapter provides mandatory, advisory and permissive standards as defined in Index 82.1. The Chief, Division of Design is responsible for approving exceptions to all mandatory standards (**boldface** text) unless delegated as noted in Index 82.2(1). District Directors are responsible for approving exceptions to all advisory standards (indicated by underlining text) as discussed in Index 82.2(2). All other guidance in this Chapter pertaining to the design of planting and irrigation systems as well as when noted in the text is the responsibility of the Landscape Architecture Program. See the Project Development Procedures Manual (PDPM) Chapter 29 regarding process and procedures for approval of deviations from Landscape standards.

901.2 Cross References

- Several highway landscape architectural terms are defined in Index 62.5 of this manual.
- The PDPM contains general definitions, policies, and procedures concerning planting and conservation of vegetation and explains procedures and responsibilities for developing highway planting projects.
- The Preliminary Environmental Analysis Report (PEAR), included in the Standard

Environmental Reference, contains guidelines and responsibilities for determining scenic resources during the project development process. <http://www.dot.ca.gov/ser/pear.htm>

- Chapter 500 of the Encroachments Permits Manual contains procedures and guidelines for planting design and administering planting by others, through permit projects.
- Chapters 4-20 and 4-21 of the Construction Manual discuss materials and methods involved in erosion control and planting and irrigation. Allowable options are described for materials and work methods called for in the project specifications as well as Landscape Architect involvement during construction.
- Chapter E of the Maintenance Manual contains instructions about the maintenance of highway planting and other roadside features. Chapter C2 of the Maintenance Manual contains instructions about the maintenance of native and naturalized roadside vegetation.
- The Landscape Architecture Program's website further explains the Department's policy and provides guidance for landscape architectural work, including water conservation. The website is located at: <http://www.dot.ca.gov/hq/LandArch/>.

Topic 902 - Planting Guidance

902.1 General Guidance for Freeways and Expressways

This section provides standards and guidelines for the design of planting and irrigation systems.

Highway planting is vegetation placed for aesthetic, environmental mitigation, storm water pollution prevention, or erosion control purposes, and includes necessary irrigation systems, inert materials, and mulches.

In addition, highway planting is used to satisfy the need for headlight glare reduction, fire retardance, windbreak protection, or graffiti reduction on retaining walls and noise barriers.

- (1) *Design Considerations.* Design planting and irrigation systems to achieve a balance between aesthetics, safety, maintainability, cost-effectiveness, and resource conservation.

Plantings should respond to local community goals.

- (a) **Aesthetics.** Select planting and replacement planting to integrate the facility with the adjacent community or natural surroundings; buffer objectionable views of the highway facility for adjacent homes, schools, parks, etc.; soften visual impacts of large structures or graded slopes; screen objectionable or distracting views; frame or enhance good views; and provide visually attractive interchanges as entrances to communities.

Select and arrange regionally appropriate drought tolerant or native plant material so the design is visually and culturally compatible with local indigenous plant communities or surrounding landscape planting.

Place plants according to the perspective of the viewer. For example, compositions viewed by freeway motorists should be simplified and large scale. Compositions primarily viewed by pedestrians may be designed with greater detail.

Contour grading that preserves existing natural features and enhances existing plants should be integrated into the overall composition.

- (b) **Safety.** Planting and irrigation facilities are designed for the safety of both highway workers and the public.

To understand potential hazards to maintenance workers, designers should be familiar with Topic 706 as well as Chapter 8, "Protection of Workers", of the Maintenance Manual.

Select and locate plants to maintain sight distance and clear recovery zone distances. Planting, without exception, must not interfere with the function of safety devices (e.g., barriers, guardrail) and traffic control devices (e.g., signals and signs), shoulders and the view from the roadway of bicyclists and pedestrians.

Cluster and locate irrigation components adjacent to access gates, maintenance

vehicle pullouts, maintenance access roads or other areas away from traffic.

Highway planting projects, should incorporate design for safety concepts that include, but are not limited to, the following:

- **Access -** Provide access gates for maintenance personnel from local streets and frontage roads. Provide paved maintenance vehicle pullout areas away from traffic on high volume highways and other areas where access cannot be made from local streets and roads. Maintenance access roads provide access to the center of loop areas or other wide, flat areas.
- **Minimize Exposure to Traffic and Reduce the Need for Shoulder or Lane Closures -** Locate irrigation system components and vegetation away from shoulder areas, gore areas, and narrow island areas between ramps and traveled way to reduce the need for shoulder or lane closures, to perform pruning or other maintenance operations. Place irrigation components that require regular maintenance, such as valves and controllers outside the clear recovery zone or behind safety devices. Narrow areas and areas behind the gore should be paved.
- **Automated Irrigation -** Use irrigation systems with "smart" controllers and remote control devices to minimize worker exposure and allow for effective water management. Cluster valves and locate the cluster adjacent to maintenance vehicle pullouts, access paths or in locations accessible from outside the right of way, via access gates.
- **Median Planting -** Median planting should not be permitted on freeways. Exceptions for the planting of freeway medians are approved by the District

Director if the planting can be maintained.

- (c) **Maintainability.** Minimize maintenance intensive activities through field observation or discussion with maintenance personnel during project development. Ongoing communication between designers, landscape specialists, landscape maintenance personnel, and construction inspectors will ensure that maintenance concerns are addressed.

Select and locate plants to reduce application of herbicides.

Specify plant establishment and irrigation test periods of sufficient time to identify and resolve problems and minimize long term maintenance requirements.

- (d) **Cost-effectiveness.** The design should provide maximum long term benefit for the costs involved. Materials and methods specified should be commercial quality and closely matched to the project conditions.
- (e) **Resource Conservation.** Maximize resource conservation through the use of regionally appropriate drought tolerant or native plants, compost, mulches, nonpotable water, automated irrigation systems, remote irrigation control systems (RICS), and moisture sensors.

Irrigation systems and associated planting are to be designed in compliance with the Model Water Efficient Landscape Ordinance (MWELo).

Highway planting should be able to withstand roadside conditions and become established on limited water with minimal maintenance. Planting designs are to account for life-cycle costs including limited maintenance resources.

Protect and preserve trees and vegetation to the maximum extent feasible during the planning, design and construction of transportation projects.

Use native species throughout the transportation system, where appropriate.

Section 130 of the Surface Transportation and Uniform Relocation Act requires at least one quarter of one percent of funds expended for a landscaping project on the Federal Aid System be used to plant native wildflowers. Additional information can be found in the FHWA manual "Roadside Use of Native Plants."

902.2 Sight Distance and Clear Recovery Zone Standards for Freeways and Expressways

Sight distance and safety are of primary importance, and are not to be subordinate to aesthetics. Applicable minimum sight distance standards are set forth in Topic 201 Sight Distance and Topic 405 Intersection Design Standards.

Two types of plant setbacks affect the placement of landscape elements:

- To keep the continuous length of highway ahead visible to the driver (sight distance).
- To keep the clear recovery zone free of physical obstructions.

(1) *Sight Distance Plant Setbacks.* Sight distance limits are measured from the edge of traveled way to the outside edge of the mature growth. Plant setback is measured from the edge of traveled way to the face of tree trunk or face of shrub foliage mass. Care must be taken to ensure that future growth will not obstruct sight distance.

Proposed mature planting should maintain sight distance required by the design speed of the facility. In cases where, due to geometric restrictions, the existing freeway facility does not provide 80 miles per hour sight distance, no further reduction should be caused by planting.

For interchanges, all planting must provide ramp and collector-distributor road sight distance equal to or greater than that required by the design speed criteria with a minimum provision of sight distance for 40 miles per hour. At points within an interchange area where ramp connections or channelization are provided, plantings must be clear of the shoulders and sight line shown in Figure

504.3J, Location of Ramp Intersections on the Crossroad.

Particular attention should be paid to planting on the inside of curves in interchange loops, in median areas, on the ends of ramps, and on cut slopes so that shoulders are clear and designed sight distances are retained for vehicles, bicycles and pedestrians. See Index 902.3.

Sight distance requirements restrict the height of plants or the horizontal distance of plants from the traveled way. Low growing plants may be planted within the plant setback distance as long as the requirements for sight distance are met as discussed in Index 201.6 and illustrated in Figure 201.6. Taller growing plants are to be placed beyond these plant setbacks. In interchange areas, generally, from the edge of traveled way, a 50-foot horizontal clearance within the loops is considered as the sight distance plant setback for trees and shrubs that will grow above a 2-foot height.

- (2) *Clear Recovery Zone.* The clear recovery zone provides an area for errant vehicles to potentially regain control. For tree setback purposes, large trees are defined as plants which at maturity, or within 10 years, have trunks 4 inches or greater in diameter, measured 4 feet above the ground. Examples of large tree species are Coast Redwood (*Sequoia sempervirens*), Coast Live Oak (*Quercus agrifolia*) and Deodar Cedar (*Cedrus deodora*).

On freeways and expressways, including interchange areas, there should be 40 feet or more of clearance between the edge of traveled way and large trees; however, a minimum clearance of 30 feet must be provided. Special considerations should be given to providing additional clearance in potential recovery areas. The 30-foot distance is measured horizontally from the edge of traveled way to the face of the tree trunk. Large trees may be planted within the 30-foot limit where they will not constitute a fixed object; for example, on cut slopes above a retaining wall or in areas behind guardrail,

which has been placed for reasons other than tree planting.

Exceptions to the 30-foot tree setback may also be considered on cut slopes which are 2:1 or steeper or where there are physical barriers such as retaining walls. The minimum tree setback in these cases should be 25 feet.

Offset distances greater than 30 feet should be provided at locations such as on the outside of horizontal curves and in the vicinity of ramp gores.

Large trees should not be planted in unprotected areas of freeway or expressway medians with the possible exception of separated roadways with medians of sufficient width to meet the plant setback requirements for tree planting.

Small trees are those with smaller trunks or plants usually considered shrubs, but trained in tree form which would not develop 4-inch diameter trunks within 10 years. Examples of small trees are Crape Myrtle (*Lagerstroemia indica*), and Bottle Brush trained as a standard (*Callistemon* sp.).

902.3 Planting Guidance for Large Trees on Conventional Highways

When proposing large trees for conventional highways the mature size, form, and growth characteristics of the species should be considered. Select and locate large trees to maintain a minimum vertical clearance of 17 feet from the pavement to the lower foliage of overhanging branches over the traveled way and shoulder to provide visibility of highway signs, features, and appurtenances. Select and locate large trees to maintain a minimum vertical clearance of 8 feet from the sidewalk to the lower foliage of overhanging branches for pedestrian passage. Do not select tree species that will require regular pruning at maturity to maintain these clearances.

Large trees must not restrict sight distance requirements.

Large trees must not visually restrict existing signs and signals.

Large trees planted in conventional highways are to

Table 902.3
Large Tree Setback Requirements on Conventional Highways

ROADSIDE			
Condition	Posted Speed (mph)		
	≤ 35	40 – 45	> 45
With curb	<u>18" Min. from curb face</u>	30' Min from ETW	
With barrier	<u>Min. deflection distance from barrier face (barrier type specific)</u>	<u>Min. deflection distance from barrier face (barrier type specific)</u>	
Without curb or barrier	30' Min from ETW		
MEDIAN ^{(1), (2)}			
Condition	Posted Speed (mph)		
	≤ 35	40 – 45	> 45
With curb	5' Min. from curb face	Not Allowed	
With curb in Main Street context; where median width of 12' is not feasible and trees are a part of a community's transportation plan to improve livability that also includes transportation features for traffic calming through physical design such as modifying intersections or relocating traffic lanes to make space for bike lanes, sidewalks and landscaping. See the Department's "Main Street, California" document for more information.	18" Min. to 5' from curb face if approved by the District Director	Not Allowed	
With barrier	Concrete Barrier: 18" Min. from face of barrier Other Barrier: Min. deflection distance for barrier type, 18" Min.		Allowed if approved by the District Director
Without curb or barrier	Not Allowed		

Notes:

- (1) Trees in the median shall be located at least 20 feet from manholes.
- (2) Trees in the median shall be located at least 100 feet from the longitudinal end of the median.

comply with the requirements in Table 902.3. All distances are measured from the frame of reference specified in Table 902.3 to the face of the tree trunk. See the District Landscape Architect for plant selection, plant setback, and spacing consistent with this guidance.

See Index 305.1(2) for median guidance on conventional highways.

902.4 Planting Procedures, Selection and Location

- (1) *Design Procedures.* An overview of the project development process is covered in the Project Development Procedures Manual.
- (2) *Plant Selection.* Plants should be tolerant of local environmental conditions such as sunlight, aspect, water availability, temperature, soil, water quality, air quality, and wind, as well as proven to be durable adjacent to highways and in transportation facilities. California native plants should be incorporated into the design, taking into account local plant communities and species availability, to the maximum extent feasible.

Plants should have the proper growth rate, longevity, size, and appearance for their intended uses. Wherever feasible, trees should be used to create the main structure of the planting composition. Plants should not require regular, ongoing maintenance other than irrigation.

A diversity of plant material should be chosen. Monoculture planting is discouraged.

Drought tolerant plants which will have the greatest chance of survival if water were to become unavailable should be selected. Species must be suitable for the project site.

If plant tolerances are questionable, the species should be avoided or used on a limited experimental basis.

Trees generally recognized to be brittle, susceptible to disease, or that increase in size by suckering, should not be selected.

Plants with edible or attractive fruits, berries or nuts should not be selected.

When appropriate, planting projects must include California native wildflowers as an integral and permanent part of the planting design. The Project Development Procedures Manual discusses wildflower requirements.

- (3) *Plant Location.* When locating plants, the mature size, form, and characteristics of the species should be considered, particularly for safety of maintenance workers and the traveling public, and long-term maintenance costs.

Plants should be located so that pruning will not be required. Trees should not be planted under overhead utilities or structures.

Plants should be located so that they will not obscure existing billboards, or on-premise business identification signs for a distance of 500 feet from the billboard sign.

Plants should be located so that they will not obscure pedestrians and bicyclists at intersections or other conflict points.

Plants with similar water requirements should be grouped for irrigation purposes.

Plants with thorns or known to be poisonous to humans and animals, (e.g., rose, oleander), should not be planted adjacent to sidewalks, bikeways, areas used for grazing animals, equestrian activities, with high public exposure, or where children have access to the planting. Designers should be aware of State and local restrictions on the planting of certain species in or adjacent to specified areas. Contact District Landscape Architect for further information.

In areas subject to frost and snow, plantings should not be located where they will cast shade and create patches of ice on vehicle or pedestrian ways.

- (4) *Planting on or Near Walls.* Vine planting should be included with all sound barrier projects to reduce the potential for graffiti and to soften the appearance of the wall. If retaining walls or sound barriers are located within the clear recovery zone (see Index 902.2), plants may be placed behind the walls and be allowed to grow over (or through) the wall, or plants may be placed in front of the

wall, but they must be behind a concrete safety shaped barrier that is placed to shield something other than plants. Plants are not permitted on concrete safety shaped barriers on the traffic side, unless an exception is granted from the Division of Traffic Operations and all of the following requirements are met:

- (a) Only vines which have a natural tendency to cling to noise barriers or retaining walls may be planted on the traffic side of barriers. Support structures on walls should not be used. The vines must readily adhere to the barriers. No shrubs or ground cover will be allowed. Vines such as Creeping Fig (*Ficus pumila*) and Algerian Ivy (*Hedera canariensis*) will not be allowed due to their habit of peeling off hard surfaces at maturity.
- (b) Plant basins must be depressed and minimal in size. Ground surface irregularities must be insignificant or nonexistent.
- (c) Each plant must be individually irrigated. The plants should not encroach onto the shoulder or create sight distance problems.

The Maintenance Unit should be consulted as vines planted on walls may require maintenance access for pruning. See Index 1102.7 for maintenance considerations in noise barrier design.

- (5) *Planting of Vines on Bridge Structures.* Vines should not be planted where they might grow over any portion of the bridge structure. When the regular inspection of bridge structures is required and where rapid visual inspection of these structures is required in areas of high seismic activity, the planting of vines on bridge structures or columns is not permitted. There are certain conditions such as low average daily traffic, high redundancy in the substructure, etc. where exceptions from Structure Maintenance may be granted, after all risk vs. benefit factors are considered, to plant vines.
- (6) *Planting in Vicinity of Airports and Heliports.* All plants must not exceed the height

restriction standards contained in Topic 207 of this manual. Mature plant height must be used to determine if the plant(s) will be considered an obstruction to navigable airspace.

902.5 Irrigation Guidelines

- (1) *General.* Irrigation systems and components should be designed to conserve water, minimize maintenance, minimize worker exposure to traffic, and sustain the planting. The design should be simple, efficient, and straight forward. Irrigation concepts utilized should conform to local water conservation goals.

Whenever available, water sources should be nonpotable, e.g., reclaimed or untreated water sources, consistent with quality and health standards, and the cost should be justified (see the Project Development Procedures Manual for cost guidelines). Water quality should be considered when selecting components and designing the system.

Standard, commercially available irrigation components should be used and special features should not be specified unless they are required to solve unique problems of the site.

Security measures, such as locking cabinets, enclosures and valve boxes should be provided.

Potential damage from pedestrians or vehicles should be considered when selecting and locating all irrigation components. Irrigation components such as controllers, valves, backflow preventers, and booster pumps are to be placed away from gores, narrow areas, decision points, and preferably located behind barriers or shielded by a structure.

- (2) *Valves and Sprinklers.* Irrigation systems should be designed for automatic operation. When systems are temporary or will be used infrequently, manual, battery, solar or timer-operated valves may be used.

Control valves are to be in manifolds where practical and a ball valve must be provided.

When appropriate, trees and shrubs, spaced more than 10 feet on center, are to be individually watered.

Overhead irrigation systems, e.g., impact or gear driven sprinklers, should be primarily used for irrigating low shrub masses, ground cover and for establishing native grasses. Trees in overhead irrigated ground cover areas should receive supplemental basin water. Sprinklers should be appropriate for local wind and soil conditions. Sprinklers should be selected and placed to avoid spraying paved surfaces. Sprinklers, other than pop-up systems, subject to being damaged by vehicles, bicyclists, or pedestrians should be relocated or provided with sprinkler protectors, flexible risers, or flow shutoff devices. Fixed risers should not be placed adjacent to sidewalks and bikeways. Sprinkler protectors should be used on pop-up sprinklers and quick coupling valves adjacent to the roadway.

- (3) *Controllers.* Irrigation controllers are to be easily accessible, located in enclosures, protected from vehicular traffic, and in an area with good lighting and visibility to oncoming traffic. Controllers must not be located near shoulders, in or near dense shrubbery, or in the path of the spray of sprinklers.
- (4) *Backflow Preventers.* The use of reduced pressure principle backflow devices are required for highway planting projects. Master remote control valves should be used at all pressured water sources directly downstream of the backflow preventers. Backflow preventers should be located in enclosures.
- (5) *Booster Pump Systems.* When local agency water pressure is insufficient, booster pumps may be included in the irrigation design. Design of a booster pump system should be coordinated with DES-SD, Office of Electrical, Mechanical, Water and Wastewater Engineering (OEMW&W). After the irrigation system has been designed such that all branches have close to equal flowrate requirements, the booster pump system design request should be prepared including flowrate and discharge pressure needed for the pump,

the availability for power distribution, and maintenance access to the pump site. OEMW&W will either design the booster pump system, (including the equipment pad, enclosure, valves and piping, pump equipment, and pump control equipment) or recommend an off-the-shelf booster pump package.

Topic 903 - Safety Roadside Rest Area Standards and Guidelines

903.1 Minimum Standards

The following standards generally represent minimum values. When consistent with sound judgment and in response to valid concerns, variations may be considered. Standards lower than those indicated herein may not be used without approval of the Principal Landscape Architect, Landscape Architecture Program. See Chapter 29 of the Project Development Procedures Manual (PDPM) for process and procedures for approval of deviations from standards.

The Division of Design is responsible for approving nonstandard geometric design as discussed in Topic 82 and Index 901.1. The Design Reviewer and Coordinator should be involved in reviewing the geometric features for the design of the on and off ramps of safety roadside rest areas. Structural sections and drainage should be designed in accordance with the standards contained in this manual.

903.2 General

Safety roadside rest areas should be designed to provide safe places for travelers in automobiles, commercial trucks, recreational vehicles, and bicycles where not prohibited, to stop for a short time, rest and manage their travel needs. Safety roadside rest areas may include vehicle parking, bicycle parking, picnic tables, sanitary facilities, telephones, water, landscape tourist information, traveler service information facilities and vending machines. Safety roadside rest areas should be provided at convenient intervals along the State highway system to accommodate traveler needs.

Safety roadside rest areas should comply with State and Federal codes and regulations that address buildings, electrical work, plumbing, lighting,

drinking water, wastewater treatment discharge, grading, storm water discharge, hazardous material containment and disposal, energy conservation, accessibility for persons with disabilities, and environmental protection and mitigation.

Safety roadside rest areas should be designed for cost effective and efficient maintenance. High quality, durable and easily cleanable materials should be used to accommodate the heavy use that rest area facilities receive. Replaceable components, such as mirrors, sinks, signs, and lighting fixtures, should be products that will be readily available during the lifetime of the facility. Crew rooms and storage space for cleaning supplies, tools and equipment should be provided in appropriate locations, away from direct public view. Maintenance access must be provided to plumbing, sewer, electrical, and equipment to facilitate inspection and repair.

The freeway interchange should accommodate, or be improved to accommodate, the volume and geometric movements of anticipated traffic. The safety roadside rest area should be within one-half mile of the freeway.

Auxiliary parking lots include parking areas and restrooms provided by or jointly developed and operated by partners (such as existing or new truck stops, or at other highway oriented commercial development). These are for longer-duration stops and overnight parking, primarily for commercial vehicle operators. These facilities are located outside of freeway right of way, within one-half mile of the freeway.

903.3 Site Selection

(1) *Need.* New safety roadside rest area and auxiliary truck parking sites should be consistent with the needs identified in the current Safety Roadside Rest Area System Master Plan. Proposed locations identified on the Safety Roadside Rest Area System Master Plan, available from the Landscape Architecture Program website, are approximate only. Actual sites may be located within several miles in either direction from the location indicated on the Safety Roadside Rest Area System Master Plan. More than one alternate site should be identified and analyzed before selecting a

preferred site. When offering potential sites for joint economic development proposals, it is best to allow for as many acceptable alternative sites as possible.

- (2) *Spacing.* New safety roadside rest area sites should be located per the current Safety Roadside Rest Area System Master Plan.
- (3) *Access.* Safety roadside rest areas located on a freeway or a highway of four lanes or more, should be planned as a pair of units, each unit serving a separate direction of traffic. Access (ingress/egress) should be by means of direct on and off ramps from the freeway or highway. Required minimum distances should be accommodated between existing and proposed ramps, in accordance with Chapter 500.

Federal law and regulations prohibit direct access from the freeway to commercial activities.

- (4) *Right of Way Requirements.* A safety roadside rest area unit may require 10 to 15 acres of right of way. Potential negative impacts to prime agricultural land, native vegetation, natural terrain, drainage and water features should be considered when identifying potential sites for rest areas. Consider sites where natural vegetation has already been disturbed and where rest area development may facilitate restoration.

Ideally, the Department should own safety roadside rest area right of way in fee simple.

However, it may be necessary or desirable for safety roadside rest areas to be located on land owned by other State, Federal or tribal entities. When seeking right of way agreements or easements, consider possible partnerships with the entity landowners that may facilitate right of way acquisition or project acceptance. The opportunity to cooperate on the development of integrated information, interpretive or welcome centers may be favorable to another entity.

- (5) *Economic Factors.* Right of way cost may be a significant factor in site selection. Advance protection or acquisition of right of way should be considered when planning and

programming future safety roadside rest area projects.

The impact of safety roadside rest areas on local tourism and economic development should be considered, addressed, and discussed. Stakeholders who may consider partnering to develop or operate the safety roadside rest area should be part of this discussion.

903.4 Facility Size and Capacity Analysis

Safety roadside rest area parking and restroom capacity should be designed to accommodate the anticipated demand in the design year (20 years from construction). When feasible, the design may allow the parking area to be expanded by 25 percent beyond the 20-year design period.

If budget prevents the full facility from being constructed initially, a master site plan should be developed that indicates the planned footprint of parking and rest rooms to accommodate anticipated demand. Areas designated for future expansion should be kept free of development, including underground utilities.

Safety roadside rest area expansion should not excessively diminish the scenic and environmental qualities of the existing site. If it is impractical to expand an existing rest area because of cost and site conditions, consider strategies for increasing capacity in the vicinity, such as relocation of the rest area, construction of an auxiliary parking facility, or construction of an additional safety roadside rest area.

(1) *Stopping Factor.* The process for estimating required parking capacity begins by calculating the percentage of daily traffic that is expected to stop at the safety roadside rest area. The Division of Traffic Operations provides data on annual average daily traffic (AADT) for State highway mainlines and ramps. The average daily ramp count for a safety roadside rest area, when divided by the mainline AADT, provides a percentage stopping factor.

$$\frac{\text{Ramp Count}}{\text{Mainline AADT}} = \text{Stopping Factor (\%)}$$

The calculated stopping factor for an existing rest area may not indicate the full demand for

a facility. Overcrowded conditions at a rest area during weekends and holidays may discourage many travelers from stopping. Nevertheless, this method provides a reasonable estimate of the rough percentage of vehicles that stop at a rest area. Stopping factors typically range from 1 percent on high volume freeways to 35 percent on remote highways.

A stopping factor cannot be directly calculated for a new safety roadside rest area; however, an estimate may be derived from existing safety roadside rest areas of similar size and situation. The type of highway traffic, the remoteness of the site, and the availability of other traveler services should be considered. Stopping factors for new safety roadside rest areas generally range from about 10 percent to 15 percent of mainline traffic.

(2) *Number of Visitors.* The number of vehicles entering a safety roadside rest area during an average day may be estimated by multiplying the mainline AADT by the stopping factor.

The number of visitors using a safety roadside rest area during an average day then may be estimated by multiplying the number of vehicles per day by an average vehicle occupancy of 2.2 people.

$$\begin{aligned} & \text{Mainline AADT (Year of Traffic data)} \\ & \times \text{Stopping Factor (\%)} \times 2.2 \\ & = \text{Total Visitors Per Day} \end{aligned}$$

To determine the 20-year design-need, it is necessary to apply a traffic-growth factor to the results. Generally, 3 percent compounded 20-year growth may be estimated by multiplying the number of visitors by a factor of 1.8.

$$\begin{aligned} & \text{Mainline AADT} \times \text{Stopping Factor (\%)} \\ & \times 2.2 \times 1.8 \\ & = \text{Total Visitors Per Day (Year of} \\ & \text{Traffic Data)} \end{aligned}$$

(3) *Number of Vehicle Parking Spaces.* The total number of parking spaces for all vehicle types may be estimated by multiplying the Peak Hour Traffic (see the Division of Traffic Operations website) by the stopping factor, and dividing the result by the number of times

the parking space is expected to turn over in one hour. Multiply by a factor of 1.8 to include the compounded 20-year growth.

Most visitors in automobiles stay about 10 minutes to 20 minutes. Some, however, will nap or sleep for longer periods. The California Code of Regulations allows travelers to stay up to 8 hours at each safety roadside rest area. For design purposes, it is common to assume a 20-minute stay for all types of vehicles (assume up to 6 hours, extended stay, for commercial truck drivers). That equals 3 turnovers of each parking space each hour.

$$\frac{\text{Peak Hour} \times \text{Stopping Factor (\%)} \times 1.8}{3 \text{ Turnovers per hour}} = \text{Total Parking Spaces (Design Year)}$$

- (4) *Automobile/Long Vehicle Split.* Consider the percentage of commercial trucks in the mainline traffic when determining the appropriate ratio of automobile parking spaces to long-vehicle parking spaces. Typically, one third of the total parking is devoted to long vehicles (commercial trucks, transit, automobiles with trailers and recreational vehicles). On certain goods-movement routes, truck traffic can account for half of the vehicular traffic at certain rest areas (consult with District Traffic Operations). For these highly commercial route segments, consider the potential for auxiliary parking facilities to satisfy the long duration stopping needs of commercial drivers at off-line parking locations.
- (5) *Bicycle Parking.* On highways where bicycling is not prohibited, bicycle parking should be provided reasonably near businesses, shopping or other amenities. Consult the District Bicycle Coordinator for information on placement, capacity, and design requirements for bicycle parking.
- (6) *Maximum Parking Capacity.* The maximum parking capacity for a safety roadside rest area unit should not exceed 120 total vehicular parking spaces. Larger facilities tend to lose pedestrian scale, context sensitivity and environmental qualities appropriate for a restful experience. If more than 120 vehicular

parking spaces are needed, it is advisable to consider the development of additional safety roadside rest areas as identified on the Safety Roadside Rest Area System Master Plan, or development of an auxiliary parking facility. Site conditions may limit the amount of parking that is practical to build. If construction or enlargement of parking areas to meet anticipated demand will significantly diminish the environmental character of the site, the quantity of parking should be reduced as appropriate.

Sites for auxiliary parking facilities should be chosen for their suitability in accommodating large numbers of commercial trucks for longer stays (up to 8 hours). Auxiliary parking facilities are not limited to 120 spaces; however, the amount of parking should be appropriate for the site and its surroundings.

- (7) *Restroom Capacity and Fixture Counts.* Restroom fixture counts (water closets, urinals for men's rooms, and lavatories) are developed by the Division of Engineering Services-Transportation Architecture, and based upon average daily visitor and peak hour visitor data provided by the District. The quantity of fixtures provided for men's rooms should be divided equally among water closets, urinals and lavatories. The quantity of water closets for women's rooms should be 1 to 1.5 times the combined quantity of toilets and urinals provided for men. Restroom facilities should be designed to accommodate visitor use during the cleaning of restrooms. When existing restrooms are replaced as part of rehabilitation projects, it is preferable that the 20-year design need be constructed, even when expansion of parking facilities is deferred. Restroom facilities must be designed and constructed to be accessible to persons with disabilities in accordance with all applicable State and Federal law.

903.5 Site Planning

- (1) *Ingress and Egress.* For safety and convenience, ingress to the safety roadside rest area, circulation within the facility and egress should be simple, direct and obvious to the traveler. See Topic 403 regarding the principles of channelization.

Rest areas designed for freeways shall have standard freeway exit and entrance ramps, in accordance with Chapter 500. Projects to rehabilitate or modify existing ramps, roads, and parking lots must address any requirement to upgrade geometrics to current design standards. Safety roadside rest areas on expressways and conventional highways should be designed with standard public road connections and median left-turn lanes, according to Topic 405.

The minimum distance between successive exit ramps on collector-distributor roads into rest areas should be 600 feet. One-way vehicular circulation should be provided through the safety roadside rest area to reduce wrong-way reentry to the freeway. Re-circulation of traffic within the parking lot is acceptable if provisions are made to discourage wrong-way traffic. Travelers should be guided towards the proper exit at each decision point along internal roads and parking aisles by the angle of intersection and the placement of curbs, pavement markings, and signs.

If the highway will ultimately be a freeway, the design should accommodate future construction. Two-way ingress/egress roads, if used, should be a minimum 32 feet wide. When a rest area or auxiliary parking facility is developed outside the freeway right of way at an interchange location, the interchange ramps, bridges and general geometric design should be capable of accommodating the volume of traffic anticipated and the turning movements of commercial trucks. Geometric and structural improvements should be completed prior to public use of the safety roadside safety roadside rest area or parking facility.

Whenever possible, ingress maneuvers should utilize simple and direct movements. Egress may be more complex, if necessary, as travelers are more rested and better prepared for a circuitous route to the freeway or highway. Provide clear signage for travelers as they approach and depart the rest area.

Travelers entering a safety roadside rest area must be directed to the proper parking area -

automobiles (cars, vans, motorcycles), bicycles, or long-vehicles. Where practical, provide ample ramps and transitions, good sight distance, and well-placed signs and pavement markings preceding the point where vehicle types separate. Avoid locating potential distractions (non-traffic-control signs, plantings, vehicle pullouts, dumpsters, artwork, etc.) at or preceding this point.

Within a safety roadside rest area, there are intersections and other points of conflict where design layout, signage, pavement markings and visibility must be carefully considered. One of these points is where long vehicle traffic, bicycle, and automobile traffic merge prior to egress from the safety roadside rest area. Consider the speed and angle at which the traffic types will merge. Avoid configurations where one type of traffic is allowed to gain excessive speed preceding a merge with slow moving traffic. Curvilinear road layout, narrow roads and landscaping can be used to manage traffic so that merging is done at slow and relatively similar speeds.

The angle of intersection should allow good visibility of oncoming traffic. Avoid blocking intersection sight lines with landscaping, signs and other elements.

Assess and improve, as necessary, ramp lengths, radii and superelevation, parking aisle widths, parking stall dimensions, and bicycle parking when rehabilitating a safety roadside rest area. When the scope of work is limited to routine pavement maintenance, such as minor repairs, seal coats and striping, or work on building, sidewalks, utilities and landscaping, upgrading to current design standards may be deferred.

- (2) *Layout.* Roads, parking areas and associated earthwork largely define the layout of a safety roadside rest area. Roads and parking areas should be arranged to fit the terrain, views and site configuration. If the site has few physical constraints, roads and parking areas should be designed with generous curves and curvilinear parking to help avoid circulation conflicts. If the site is heavily wooded, roads and parking should be designed to retain the healthiest and most attractive trees and tree groupings.

Walking distance from the most remote parking space to restrooms should not exceed 350 feet.

Bicycle parking should be located in a safe area.

To maintain visual quality and avoid environmental damage to soils, vegetation and water quality, paved service roads should be provided for maintenance access to service facilities. Service roads should be 10 feet to 12 feet wide.

(3) *Grading and Drainage.* Grading should be designed to accommodate and integrate the required development with as little disturbance to the site as practical. Drainage should be designed in accordance with Chapter 800 through 860. Grading and drainage should be harmonious with natural landforms and follow the direction of existing slopes and drainage patterns. Cuts and fills should be shaped and rounded to blend with existing land forms, and the revised terrain should complement the layout of parking areas and sidewalks.

(4) *Parking Areas.* Ramps, interior roads and parking areas should be designed to encourage safe and orderly traffic movement and parking. These areas should be well defined and when appropriate include the use of concrete curbs and striping.

The design of all roads, aisles, parking spaces and parking lot islands should ensure that commercial truck maneuvers can be accommodated without damage to curbs, sidewalks, pavement edges or parked vehicles. See Topic 407 for truck and bus turning template guidance.

Provide one dedicated parking space for use by the California Highway Patrol (CHP). The CHP space should be located in an area that provides maximum visibility to the public. If a CHP drop-in office is planned, the CHP space should be visible from the office location. Provide a sign and pavement markings to designate the CHP space. A sign advising “Patrolled by Highway Patrol” should be placed on the freeway exit sign preceding each rest area.

Parking facilities are to be designed accessible to all modes of travel and are to conform to California MUTCD and DIB 82 guidance. Designated accessible parking spaces must be provided for automobiles and vans. As space permits and need requires, one accessible parking space for long vehicles may be provided at each rest area unit. Refer to Chapters 600 through 670 for pavement structure guidance.

(5) *Pavement.* Pavement for ramps, roads and parking should be designed in accordance with Chapters 600 through 670. Parking lots may be constructed of flexible or rigid pavement. Rigid pavement has the advantage of being resistant to deterioration from dripping fuel and antifreeze.

**Table 903.5
Vehicle Parking Stall Standards**

Vehicle Type	Min Stall Width (ft)	Aisle Width (ft)	Aisle Location
1 Auto	9	5	Passenger side
2 Autos	9	5	Between stalls
1 Van	9	8	Passenger side
1 Van/ 1 Auto	9	8	Between stalls
1 long vehicle	12	8	Passenger side
2 long vehicles	12	8	Between stalls

(6) *Signage.* Standard reflectorized signs should be placed along the roadside to inform and direct travelers as they approach a safety roadside rest area. A roadside sign should be placed one mile in advance of each safety roadside rest area that indicates the distance to

that rest area and to the next rest area beyond. In remote areas an additional sign may be placed in advance of a safety roadside rest area indicating the distance to the facility. Additional panels may be included on or near this sign to inform travelers of the availability of vending machines, recreational vehicle waste disposal stations, traveler information, wireless internet or other special services. A directional sign should be placed at the safety roadside rest area ingress ramp. Standard reflectorized traffic control signs should be used within the rest area for all traffic guidance. These signs may be enhanced with aesthetic backing or frames. Non-traffic signs may be of customized design, provided they are easy to maintain or replace should they be damaged or stolen.

Freestanding signs should be placed in safety roadside rest areas only to provide traveler direction. However, a welcome sign indicating the safety roadside rest area name may be placed within the pedestrian portion of the rest area. Welcome signs should not be placed along ramps or at traffic decision points. Welcome signs must not be placed within the clear recovery zone of the highway or ramps. Informational signs indicating use regulations, anti-litter regulations, reclaimed water use, safety roadside rest area adoptions, maintenance crews presence/hours, proximity/use of agricultural crops, scenic highways designation, environmental features, etc., should be placed in kiosks, display cases, or interpretive displays designed for pedestrian viewing (see DIB 82 for guidance on exhibits).

- (7) *Walkways.* It is important to provide a clearly defined and ADA compliant path of travel for pedestrians. Primary walkways should be located to direct users from automobile, bicycle, and long-vehicle parking areas to core facilities and restroom entrances. See DIB 82 for further information on accessibility requirements.

Walkways should be a minimum 10 feet wide. Steps should be avoided. Sidewalks in front of automobile parking spaces should be a minimum of 12 feet wide to compensate for

the overhang of automobiles where wheel stops are not provided. Tree wells smaller than 4 feet in dimension should not be placed in sidewalks or pedestrian plazas to avoid displacement of pavement by tree roots. Trees adjacent to walkways are to provide a minimum clearance of 8 feet from pavement to lower foliage.

Accessible paths of travel must be provided to restrooms and other pedestrian facilities, including picnic shelters, picnic tables, benches, drinking fountains, telephones, vending machines, information kiosks, interpretive displays, and viewing areas. The path of travel from designated accessible parking to accessible facilities should be as short and direct as practical, must have an even surface, and must include curb ramps, marked aisles and crosswalks, and other features, as required to facilitate visitors with wheelchairs, walkers and other mobility aids. The Department of General Services, Division of State Architect, as well as the California Department of Transportation enforce the California Building Code (Title 24) for the various on-site improvements. Many of these design requirements are contained in DIB 82 for exterior features, but many other design requirements are not in DIB 82 and still must be followed. The Division of Engineering Services - Transportation Architecture may be consulted for assistance.

- (8) *Service Facilities.* Service facilities including, crew rooms, equipment storage rooms, dumpster enclosures, service yards, and utility equipment, can be distracting and unattractive to rest area users. Service facilities should be aesthetically attractive, separated and oriented away from public-use areas (restrooms, pedestrian core and picnic areas).

903.6 Utility Systems

Utility systems should be designed in conformance with Title 24 Energy Requirements of the California Code of Regulations (State Building Code), and other applicable State and Federal requirements.

- (1) *Electrical Service.* Electrical power systems should be designed to accommodate the

demands, as applicable, of outdoor lighting (ramps, parking areas, pedestrian walkways and plazas), water supply systems (pumps, pressure tanks, irrigation controllers), restrooms (lighting, hand dryers), pedestrian facilities (lighting, water chillers, telephones, wireless internet, kiosks), crew room (lighting, heating, air conditioning, refrigerator, microwave), CHP drop-in office (lighting, heating, air conditioning), and vending (lighting, vending machines, change machine, storage-room air conditioning).

Primary electrical power sufficient for basic safety needs should be supplied by conventional power providers. Supplemental power may be provided using innovative technologies such as solar panels or wind generation or conventional means, such as backup generators. Consider security, public safety and environmental protection when considering the type of fuel and fuel storage facilities for electrical generation. Provide vehicular access to fuel storage facilities for refueling, and include fencing and gates as necessary to prevent access by the general public.

- (2) *Water.* Water supply systems should be designed to accommodate the 20-year projected demand and to handle the peak flow required for restroom fixtures and landscape irrigation. Pumps, pressure tanks, chlorinators and associated equipment should be located outside of pedestrian use areas and screened from view. Enclosures should be provided for water supply equipment to discourage vandalism and minimize the appearance of clutter. Water lines beneath parking areas, pedestrian plazas and the highway should be placed in conduits. Maintain appropriate distance between wells and wastewater disposal facilities (applicable laws should be followed). Potable water must be provided to sinks, drinking fountains, exterior faucet assemblies and pet-watering stations. Untreated or non-potable water may be used for toilets and landscape irrigation. Irrigation systems should be isolated from the general water system using appropriate backflow prevention devices.

- (3) *Wastewater Disposal.* Wastewater disposal facilities should be designed to handle the peak sewage demand. Waterborne sewage disposal systems should be provided. Structures Design will arrange for soil analysis and percolation tests, and upon completion of testing will obtain approval of the proposed sewage treatment system from the Regional Water Quality Control Board. Recreation vehicle waste disposal stations may be provided at rest areas where there is a recognized need and commercial disposal stations are not available.

- (4) *Telephones.* Provide locations, conduit and wiring for a minimum of three public pay telephones at each safety roadside rest area unit. To comply with accessibility laws and regulations, at least one telephone must be wheelchair accessible, at least one telephone must allow for audio amplification, and at least one telephone must include text messaging for the hearing impaired. Whenever possible, all telephones should allow for audio amplification.

Telephones should be wall or pedestal mounted, and located in pedestrian areas that are well lighted, and whenever possible, protected from rain, snow and wind. Consider placing telephones, commercial advertising displays and public information displays in close proximity. Information should be placed near telephones indicating local emergency numbers and indicating the rest area name and location. 120-volt power should be provided to operate keyboards and pedestal lighting.

Conduits and pull wires should be provided from the telephone service point to the maintenance crew room and to the California Highway Patrol (CHP) drop-in office. Provide telephone service for maintenance contractors and the CHP.

- (5) *Call Boxes.* Call Boxes generally are not placed in safety roadside rest areas.
- (6) *Telecommunications Equipment and Transmission Towers.* The Department seeks revenue from placement of wireless telecommunications facilities on State-owned

right of way. Transmission towers and associated equipment, structures and fencing should be located outside of pedestrian use areas and views. Telecommunications equipment and transmission towers should be aesthetically integrated into the site. Consider future safety roadside rest area expansion, and, when possible, locate facilities outside of areas planned for future development.

- (7) *Lighting.* Site and building lighting are to be designed in conformance with Title 24 Energy Requirements of the California Code of Regulations (State Building Code). Also refer to the Traffic Manual, Chapter 9 for further Highway Lighting guidance. For functionality and safety, rest areas should be lighted for 24-hour-a-day use. Lighting should be automatically controlled and include manual-shutoff capability. Restroom entrances and the interiors of restrooms, utility corridors, crew rooms, CHP drop-in offices and storage buildings, pedestrian plazas, primary sidewalks, crosswalks, ramps, picnic areas, kiosks, bicycle parking, and interpretive displays should be brightly illuminated. Lighting should illuminate walking surfaces and avoid strong shadows. An average level of 1 foot-candle is generally acceptable for primary pedestrian areas. Peripheral areas of the site should be lighted only where nighttime pedestrian use is anticipated. Non-pedestrian areas of the site do not require lighting.

903.7 Structures

Safety roadside rest area structures include restrooms, storage rooms, equipment rooms, crew rooms, CHP drop-in offices, picnic shelters, utility enclosures, dumpster enclosures, kiosks, arbors and other architectural elements. Safety roadside rest area architecture should be designed for a service life of approximately 20 years. Safety roadside rest areas are high-profile public works projects, which represent the State, Department and local community to millions of visitors each year. Attention to quality architectural design, construction and maintenance is warranted. Building forms, rooflines, construction materials (stone, timber, steel, etc.), colors and detailing should express the local context including history,

cultural influences, climate, topography, geology and vegetation. Structures must be designed and constructed to be accessible to persons with disabilities in accordance with all applicable State and Federal law.

- (1) *Restrooms.* Two restrooms should be provided for each gender to allow for uninterrupted public access to facilities during janitorial cleaning operations. Unisex or family restrooms may be provided to facilitate assistance by others to young children, elderly persons and persons with disabilities. These facilities are not considered part of the total capacity used, but may be counted as women's restrooms.

Entrances to restrooms should be visible from the parking area. They should be well lighted and clearly identified with signs and/or graphics. Restroom entrances should not be located in areas of dead-end circulation. Facilities intended for general public use should not be located near restroom entrances. Privacy screens at restroom entrances should allow visibility from the ground to a height of 12 inches to 18 inches above the ground. Lockable steel doors should be provided for entrances to rest rooms, storage rooms, crew rooms and CHP drop-in offices.

To deter vandalism, signs should be made of metal or other durable material and should be recessed into, or securely mounted on a wall. Signs identifying the entrance to each restroom should be clearly visible from the parking area. A sign, in English and Braille, should be placed on the building wall or on the privacy screen at each restroom entrance to identify the gender. Signs may also be provided in other languages as appropriate. A standard sign should be installed near the entrance to each restroom advising that, pursuant to Streets and Highways Code Section 223.5, a person of the opposite sex may accompany a person with a disability into the restroom. A sign should be installed near the restroom doors advising that, State law prohibits smoking in restrooms and the area within 20 feet of the restroom doors.

- (2) *Crew Room.* A maintenance crew room, separate from equipment and supply storage,

should be provided at each safety roadside rest area. When appropriate, a single crew room may be provided for a pair of safety roadside rest area units. The crew room should be heated and air-conditioned. Conduits or wiring for telephone service, by others, may be provided.

- (3) *CHP Drop-in Office.* A dedicated office and restroom should be provided for use by the CHP. Consult with the CHP to determine need. The office should be located adjacent to the pedestrian core and near the dedicated CHP parking stall. The restroom may have double entries to allow cleaning by maintenance crews; however, the CHP office should be designed to allow access only by CHP.

- (4) *Vending Machine Facilities.* Accommodations for vending machines should be considered when designing safety roadside rest areas. Vending machines may be installed with a project or installed at any other time by initiative of the California Department of Rehabilitation, Business Enterprise Program (BEP).

A storage room should be provided within 150 feet of the vending machines for storage of vended products. The safety roadside rest area project should provide conduits from the electrical service panel to the vending storage room for possible installation of air conditioning by the BEP.

- (5) *Storage Rooms or Buildings.* Storage rooms or buildings should be provided to house maintenance equipment, tools and supplies. Janitorial cleaning supplies and tools should be located in the vicinity of the restrooms, reasonably close to parking for maintenance service vehicles. Grounds-maintenance equipment and supplies should be located outside of public-use areas and views. Shelving for paper goods, cleaning supplies and other materials must be provided.
- (6) *Caretakers/Managers.* Residential facilities or offices for caretakers or managers may be included with a safety roadside rest area when prior provisions have been made for the use and staffing of such facilities. Caretakers and

managers may be employed or otherwise compensated, sponsored by others, or work as volunteers.

- (7) *Public Information Facilities.* At least 96 square feet of lighted display space should be provided at each safety roadside rest area for display of public information, such as rest area regulations, maps, road conditions, rest area closures, safety tips, and missing children posters. Space should consist of wall-mounted cases or freestanding kiosks.

903.8 Security and Pedestrian Amenities

Proper safety roadside rest area design will help ensure user safety with the installation of adequate lighting, providing accessible walking surfaces and allowing open visibility through the site. Vegetation, walls, recesses and other areas that allow concealment should not be located near restroom entrances. Site security may also include the presence of a CHP office and the use of surveillance cameras. Fences should be provided only for access control, traffic control, or safety purposes. Fencing should be designed to be as unobtrusive as practical. A 4-foot high fence must be provided between the highway and the safety roadside rest area. Perimeter fencing should be of the minimum height and design necessary. Where adjacent property is developed, more substantial fencing or screening may be required. Fencing in rural or natural areas may be required to control or protect wildlife or livestock.

Pedestrian amenities include trash and recycling facilities, pedestrian signs, pet areas and drinking fountains. Landscape architectural elements such as shade structures, kiosks, benches, seat walls, picnic tables, and other miscellaneous features should be included. Landscaping should be provided and may include areas for monuments, artwork, interpretive facilities, and informal exercise and play facilities. Newspaper and traveler coupon booklet vending machines are owned by others and placed in safety roadside rest areas by encroachment permit. Pedestrian amenities must be designed and constructed to be accessible to persons with disabilities in accordance with all applicable State and Federal law.

Wireless internet facilities may be installed in safety roadside rest areas with funding borne by the provider or others.

Coin operated binocular viewing as authorized by law is provided privately through a competitively awarded revenue-generating agreement.

Topic 904 - Vista Point Standards and Guidelines

904.1 General

New vista points should be considered during planning and design of new alignments for inclusion with the highway contract (see Index 109.3). Vista points may also be provided on existing routes. Existing vista points should be periodically inspected for needed restoration or upgrading.

The District Landscape Architect is responsible for approving site selection, concept, and design for all areas to be signed as vista points. Pavement structure and drainage should be designed in accordance with the standards contained in this manual.

Vista points should be designed to be accessible to all travelers and conform to the Americans with Disabilities Act and DIB 82.

904.2 Site Selection

Site selection is based on the following criteria:

- (1) *Quality.* A site should have views and scenery of outstanding merit or beauty. Locations on designated State scenic highways or in areas of historical or environmental significance should be given special emphasis. A site should provide the best viewing opportunities compared to other potential locations within the vicinity.
- (2) *Compatibility.* A site should be located on State highway right of way or on right of way secured by easement or agreement with another public agency. A site should be obtainable without condemnation. Sites on or adjacent to developed property or property where development is anticipated should be avoided.

- (3) *Access.* A site must be accessible from a State highway or intersecting road. A site must have adequate sight distance for safe access.

- (4) *Adequate Space.* A site must be of adequate size to accommodate the necessary features and facilities. However, development of a site can not detract from the scenic quality of the area. Adequate space should be available for earth mounding and planting to minimize the visual impact of larger facilities. Adequate space for future expansion is desirable.

904.3 Design Features and Facilities

- (1) *Road Connections.* The design of connections to vista points should be in accordance with Index 107.1. **Vista points designed for freeways shall have standard freeway exit and entrance ramps (see Chapter 500).**
- (2) *Parking.* Parking areas should be inclusive of all user modes. Parking capacity should be based on an analysis of current traffic data. However, at least five vehicle spaces should be provided. Parking should not exceed 0.025 times the DHV or 50 spaces, whichever is less. This number may be exceeded at high use trailheads. Parking stalls should be delineated by striping. Approximately one-quarter to one-third of the spaces should be allocated to long vehicles (cars with trailers, recreational vehicles, and buses). Geometrics should be such that all types of vehicles entering the vista point can safely negotiate and exit the facility. Accessible parking should be provided as discussed in Index 903.5(4) and DIB 82.

Consult the District Bicycle Coordinator for guidance on bicycle parking.

- (3) *Pedestrian Areas.* Vista points should provide a safe place where motorists can observe the view from outside their vehicles and bicyclists off their bicycles. Accessible walkways that exclude vehicles may be provided within the viewing area.
- (4) *Interpretive Displays.* An interpretive display should be provided within the pedestrian area of each vista point. The display should be appropriate to the site, both in design and content and accessible; see DIB 82 for exhibit

guidance. Display structures should not overwhelm or dominate the site, and they should be placed at the proper location for viewing the attraction.

Information should pertain to local environmental, ecological, and historical features. It should interpret the features being viewed to inform and educate the public.

Historical plaques, monuments, vicinity maps, and directions to other public facilities are examples of other appropriate informational items.

- (5) *Vending Machines and Public Information Displays.* Designers should be familiar with the provisions of the California Streets and Highways Code, Section 225-225.5. The designer should adequately consider and plan for uses and facilities that may reasonably be anticipated.
- (6) *Sanitary Facilities.* Comfort stations are usually not provided. Exceptions must be approved by the Principal Landscape Architect, Landscape Architecture Program.
- (7) *Water.* Potable water may be provided at a reasonable cost. Nonpotable water should not be provided in a vista point.
- (8) *Trash Receptacles.* Trash receptacles should be provided in each vista point. As a guide, one receptacle should be provided for every four cars, but a minimum of two receptacles should be provided per vista point. Dumpsters should not be located at a vista point.
- (9) *Signs.* Directional, regulatory, and warning signs must conform to the California MUTCD.
- (10) *Planting.* Existing vegetation, rock outcroppings, and other natural features should be conserved and highlighted. Removal or pruning of existing plants to frame the view should be held to a minimum and be directed by the District Landscape Architect. Earth mounding and contour grading may be employed to restore and naturalize the site. Planting, including erosion control, should be provided to revegetate graded areas. Plants requiring permanent irrigation should be avoided.

- (11) *Barriers.* Railings, bollards, or other appropriate barriers should be used to protect pedestrians, and discourage entry into sensitive or hazardous areas.

The design of such barriers should be sensitive to pedestrian scale and reflect the scenic character of the site.

- (12) *Other Features.* Benches, telephones, and viewing machines are optional items. Picnic tables are not to be included in vista points.

In general, the inclusion of items which do not either facilitate the viewing of the scenic attraction, or blend the vista point into its surroundings, should be avoided.

Topic 905 - Park and Ride Standards and Guidelines

905.1 General

Park and Ride facilities must be considered for inclusion on all major transportation projects that include, but are not limited to, new freeways, interchange modifications, lane additions, transit facilities, and HOV lanes. See Chapter 8, Section 7 of the Project Development Procedures Manual for additional information.

The District Park and Ride Coordinator is responsible for approving site selection. The concept and general design for Park and Ride facilities must be coordinated by the District Landscape Architect. Additional information on Park and Ride facilities can be obtained from the Headquarters Park and Ride Coordinator in the Office of System Management Operations in the Division of Traffic Operations. Additional guidance on Park and Ride facilities can be found in the AASHTO Publication "Guide for Park and Ride Facilities" (2004).

Park and Ride facilities must accommodate all modes of travel and conform to the American with Disabilities Act and DIB 82.

905.2 Site Selection

Park and Ride facilities are typically placed to enhance corridor efforts to reduce congestion, and to improve air quality usually associated with other transportation opportunities such as HOV lanes and transit. The specific choice as to location and

design should be supported by a detailed analysis of demand and the impact of a Park and Ride facility based upon these parameters:

- Corridor congestion
- Community Values
- Air Quality
- Transit Operations
- Overall Safety
- Multi-modal Opportunities

Full involvement of the project development team should be engaged in the evaluation and recommendation of Park and Ride type, classification, site and appurtenant facilities.

905.3 Design Features and Facilities

Park and Ride facilities are to be designed as multi-modal facilities. Provisions for pedestrians, bicyclists, transit, single-occupancy vehicles, and multi-occupancy vehicles are to be provided as appropriate. The local transit provider should be consulted to determine if the facility should provide connections to transit. In general, the function of the facility is to take precedent over the form of the facility; however, special consideration for the safety and security of all users is fundamental to the success of the facility.

The design of a Park and Ride facility should take into account the operations and maintenance of the facility, both in terms of effort as well as safety. Appurtenant facilities as allowed by law should be carefully evaluated and included as appropriate. Any necessary funding and agreements need to allow appurtenant facilities on site and should be in place early in the project development process.

CHAPTER 1000 BICYCLE TRANSPORTATION DESIGN

Topic 1001 - Introduction

Index 1001.1 – Bicycle Transportation

The needs of nonmotorized transportation are an essential part of all highway projects. Mobility for all travel modes is recognized as an integral element of the transportation system. Therefore, the guidance provided in this manual complies with Deputy Directive 64-Revision #1: Complete Streets: Integrating the Transportation System. See AASHTO, “Guide For The Development Of Bicycle Facilities”.

Design guidance for Class I bikeways (bike paths), Class III bikeways (bike routes) and Trails are provided in this chapter. Design guidance that addresses the mobility needs of bicyclists on all roads as well as on Class II bikeways (bike lanes) is distributed throughout this manual where appropriate.

See Topic 116 for guidance regarding bikes on freeways.

1001.2 Streets and Highways Code References

The Streets and Highways Code Section 890.4 defines a “bikeway” as a facility that is provided primarily for bicycle travel. Following are other related definitions, found in Chapter 8 Nonmotorized Transportation, from the Streets and Highway Code:

- (a) Section 887 -- Definition of nonmotorized facility.
- (b) Section 887.6 -- Agreements with local agencies to construct and maintain nonmotorized facilities.
- (c) Section 887.8 -- Payment for construction and maintenance of nonmotorized facilities approximately paralleling State highways.
- (d) Section 888 -- Severance of existing major non-motorized route by freeway construction.

- (e) Section 888.2 -- Incorporation of nonmotorized facilities in the design of freeways.
- (f) Section 888.4 -- Requires Caltrans to budget not less than \$360,000 annually for nonmotorized facilities used in conjunction with the State highway system.
- (g) Section 890.4 -- Class I, II, and III bikeway definitions.
- (h) Section 890.6 - 890.8 -- Caltrans and local agencies to develop design criteria and symbols for signs, markers, and traffic control devices for bikeways and roadways where bicycle travel is permitted.
- (i) Section 891 -- Local agencies must comply with design criteria and uniform symbols.
- (j) Section 892 -- Use of abandoned right-of-way as a nonmotorized facility.

1001.3 Vehicle Code References

- (a) Section 21200 -- Bicyclist's rights and responsibilities for traveling on highways.
- (b) Section 21202 -- Bicyclist's position on roadways when traveling slower than the normal traffic speed.
- (c) Section 21206 -- Allows local agencies to regulate operation of bicycles on pedestrian or bicycle facilities.
- (d) Section 21207 -- Allows local agencies to establish bike lanes on non-State highways.
- (e) Section 21207.5 -- Prohibits motorized bicycles on bike paths or bike lanes.
- (f) Section 21208 -- Specifies permitted movements by bicyclists from bike lanes.
- (g) Section 21209 -- Specifies permitted movements by vehicles in bike lanes.
- (h) Section 21210 -- Prohibits bicycle parking on sidewalks unless pedestrians have an adequate path.
- (i) Section 21211 -- Prohibits impeding or obstruction of bicyclists on bike paths.
- (j) Section 21400 -- Adopt rules and regulations for signs, markings, and traffic control devices for roadways user.

- (k) Section 21401 -- Only those official traffic control devices that conform to the uniform standards and specifications promulgated by the Department of Transportation shall be placed upon a street or highway.
- (l) Section 21717 -- Requires a motorist to drive in a bike lane prior to making a turn.
- (m) Section 21960 -- Use of freeways by bicyclists.
- (n) Section 21966 -- No pedestrian shall proceed along a bicycle path or lane where there is an adjacent adequate pedestrian facility.

1001.4 Bikeways

(1) Role of Bikeways

Bikeways are one element of an effort to improve bicycling safety and convenience - either to help accommodate motor vehicle and bicycle traffic on the roadway system, or as a complement to the road system to meet the needs of the bicyclist.

Off-street bikeways in exclusive corridors can be effective in providing new recreational opportunities, and desirable transportation/commuter routes. Off-street bikeways can also provide access with bridges and tunnels which cross barriers to bicycle travel (e.g., freeway or river crossing). Likewise, on-street bikeways can serve to enhance safety and convenience, especially if other commitments are made in conjunction with establishment of bikeways, such as: elimination of parking or increased roadway width, elimination of surface irregularities and roadway obstacles, frequent street sweeping, established intersection priority on the bike route street as compared with the majority of cross streets, and installation of bicycle-sensitive loop detectors at signalized intersections.

(2) Decision to Develop Bikeways

Providing an interconnected network of bikeways will improve safety for all users and access for bicycles. The development of well conceived bikeways can have a positive effect on bicyclist and motorist behavior. In addition, providing an interconnected network of bikeways along with education and enforcement

can improve safety and access for bicyclists. The decision to develop bikeways should be made in coordination with the local agencies.

Topic 1002 - Bikeway Facilities

1002.1 Selection of the Type of Facility

The type of facility to select in meeting the bicyclist's need is dependent on many factors, but the following applications are the most common for each type.

(1) Shared Roadway (No Bikeway Designation).

Most bicycle travel in the State now occurs on streets and highways without bikeway designations and this may continue to be true in the future as well. In some instances, entire street systems may be fully adequate for safe and efficient bicycle travel, where signing and pavement marking for bicycle use may be unnecessary. In other cases, prior to designation as a bikeway, routes may need improvements for bicycle travel.

Many rural highways are used by touring bicyclists for intercity and recreational travel. It might be inappropriate to designate the highways as bikeways because of the limited use and the lack of continuity with other bike routes. However, the development and maintenance of 4-foot paved roadway shoulders with a standard 4 inch edge line can significantly improve the safety and convenience for bicyclists and motorists along such routes.

- #### (2) Class I Bikeway (Bike Path).
- Generally, bike paths should be used to serve corridors not served by streets and highways or where wide right of way exists, permitting such facilities to be constructed away from the influence of parallel streets. Bike paths should offer opportunities not provided by the road system. They can either provide a recreational opportunity, or in some instances, can serve as direct high-speed commute routes if cross flow by motor vehicles and pedestrian conflicts can be minimized. The most common applications are along rivers, ocean fronts, canals, utility right of way, abandoned railroad right of way, within school campuses, or within and between parks. There may also be situations where such

facilities can be provided as part of planned developments. Another common application of Class I facilities is to close gaps to bicycle travel caused by construction of freeways or because of the existence of natural barriers (rivers, mountains, etc.).

- (3) *Class II Bikeway (Bike Lane)*. Bike lanes are established along streets in corridors where there is significant bicycle demand, and where there are distinct needs that can be served by them. The purpose should be to improve conditions for bicyclists in the corridors. Bike lanes are intended to delineate the right of way assigned to bicyclists and motorists and to provide for more predictable movements by each. But a more important reason for constructing bike lanes is to better accommodate bicyclists through corridors where insufficient room exists for side-by-side sharing of existing streets by motorists and bicyclists. This can be accomplished by reducing the number of lanes, reducing lane width, or prohibiting or reconfiguring parking on given streets in order to delineate bike lanes. In addition, other things can be done on bike lane streets to improve the situation for bicyclists that might not be possible on all streets (e.g., improvements to the surface, augmented sweeping programs, special signal facilities, etc.). Generally, pavement markings alone will not measurably enhance bicycling.

If bicycle travel is to be provided by delineation, attention should be made to assure that high levels of service are provided with these lanes. It is important to meet bicyclist expectations and increase bicyclist perception of service quality, where capacity analysis demonstrates service quality measures are improved from the bicyclist's point of view.

Design guidance that addresses the mobility needs of bicyclists on Class II bikeways (bike lanes) is also distributed throughout this manual where appropriate.

- (4) *Class III Bikeway (Bike Route)*. Bike routes are shared facilities which serve either to:
- (a) Provide continuity to other bicycle facilities (usually Class II bikeways); or

- (b) Designate preferred routes through high demand corridors.

As with bike lanes, designation of bike routes should indicate to bicyclists that there are particular advantages to using these routes as compared with alternative routes. This means that responsible agencies have taken actions to assure that these routes are suitable as shared routes and will be maintained in a manner consistent with the needs of bicyclists. Normally, bike routes are shared with motor vehicles. The use of sidewalks as Class III bikeways is strongly discouraged.

It is emphasized that the designation of bikeways as Class I, II and III should not be construed as a hierarchy of bikeways; that one is better than the other. Each class of bikeway has its appropriate application.

In selecting the proper facility, an overriding concern is to assure that the proposed facility will not encourage or require bicyclists or motorists to operate in a manner that is inconsistent with the rules of the road.

An important consideration in selecting the type of facility is continuity. Alternating segments of Class I and Class II (or Class III) bikeways along a route are generally incompatible, as street crossings by bicyclists is required when the route changes character. Also, wrong-way bicycle travel will occur on the street beyond the ends of bike paths because of the inconvenience of having to cross the street.

Topic 1003 - Bikeway Design Criteria

1003.1 Class I Bikeways (Bike Paths)

Class I bikeways (bike paths) are facilities with exclusive right of way, with cross flows by vehicles minimized. Motor vehicles are prohibited from bike paths per the CVC, which can be reinforced by signing. Class I bikeways, unless adjacent to an adequate pedestrian facility, (see Index 1001.3(n)) are for the exclusive use of bicycles and pedestrians, therefore any facility serving pedestrians must meet accessibility requirements, see DIB 82. However, experience has shown that if regular pedestrian use

is anticipated, separate facilities for pedestrians maybe beneficial to minimize conflicts. Please note, sidewalks are not Class I bikeways because they are primarily intended to serve pedestrians, generally cannot meet the design standards for Class I bikeways, and do not minimize vehicle cross flows. See Index 1003.3 for discussion of the issues associated with sidewalk bikeways.

(1) *Widths and Cross Slopes.* See Figure 1003.1A for two-way Class I bikeway (bike path) width, cross slope, and side slope details. The term “shoulder” as used in the context of a bike path is an unobstructed all weather surface on each side of a bike path with similar functionality as shoulders on roadways with the exception that motor vehicle parking and use is not allowed. The shoulder area is not considered part of the bike path traveled way.

Experience has shown that paved paths less than 12 feet wide can break up along the edge as a result of loads from maintenance vehicles.

(a) *Traveled Way.* **The minimum paved width of travel way for a two-way bike path shall be 8 feet, 10-foot preferred. The minimum paved width for a one-way bike path shall be 5 feet.** It should be assumed that bike paths will be used for two-way travel. Development of a one-way bike path should be undertaken only in rare situations where there is a need for only one-direction of travel. Two-way use of bike paths designed for one-way travel increases the risk of head-on collisions, as it is difficult to enforce one-way operation. This is not meant to apply to two one-way bike paths that are parallel and adjacent to each other within a wide right of way.

Where heavy bicycle volumes are anticipated and/or significant pedestrian traffic is expected, the paved width of a two-way bike path should be greater than 10 feet, preferably 12 feet or more. Another important factor to consider in determining the appropriate width is that bicyclists will tend to ride side by side on bike paths, and bicyclists may need adequate passing clearance next to pedestrians and slower moving bicyclists.

See Index 1003.1(16) Drainage, for cross slope information.

(b) *Shoulder.* **A minimum 2-foot wide shoulder, composed of the same pavement material as the bike path or all weather surface material that is free of vegetation, shall be provided adjacent to the traveled way of the bike path when not on a structure;** see Figure 1003.1A. A shoulder width of 3 feet should be provided where feasible. A wider shoulder can reduce bicycle conflicts with pedestrians. Where the paved bike path width is wider than the minimum required, the unpaved shoulder area may be reduced proportionately. If all or part of the shoulder is paved with the same material as the bike path, it is to be delineated from the traveled way of the bike path with an edgeline.

See Index 1003.1(16), Drainage, for cross slope information.

(2) *Bike Path Separation from a Pedestrian Walkway.* If there is an adjacent pedestrian walkway, the edge of the traveled way of the bike path is to be separated from the pedestrian walkway by a minimum width of 5 feet of unpaved material. The 5-foot area of unpaved material may include landscaping or other features that provide a continuous obstacle to deter bike path and walkway users from using both paths as a single facility. These obstacles may be fences, railings, solid walls, or dense shrubbery. Flexible delineators, poles, curbs, or pavement markers are not to be used because they will not deter users from using both paths as a single facility. These obstacles between the pedestrian walkways and bike paths are not to obstruct stopping sight distance along curves or corner sight distance at intersections with roadways or other paths.

(3) *Clearance to Obstructions.* **A minimum 2-foot horizontal clearance from the paved edge of a bike path to obstructions shall be provided.** See Figure 1003.1A. 3 feet should be provided. Adequate clearance from fixed objects is needed regardless of the paved width. If a path is paved contiguous with a continuous fixed object (e.g., fence, wall, and

building), a 4-inch white edge line, 2 feet from the fixed object, is recommended to minimize the likelihood of a bicyclist hitting it. **The clear width of a bicycle path on structures between railings shall be not less than 10 feet.** It is desirable that the clear width of structures be equal to the minimum clear width of the path plus shoulders (i.e., 14 feet).

The vertical clearance to obstructions across the width of a bike path shall be a minimum of 8 feet and 7 feet over shoulder. Where practical, a vertical clearance of 10 feet is desirable.

- (4) *Signing and Delineation.* For application and placement of signs, see the California MUTCD, Section 9B. For pavement marking guidance, see the California MUTCD, Section 9C.
- (5) *Intersections with Highways.* Intersections are an important consideration in bike path design. Bicycle path intersection design should address both cross-traffic and turning movements. If alternate locations for a bike path are available, the one with the most beneficial intersection characteristics should be selected.

Where motor vehicle cross traffic and bicycle traffic is heavy, grade separations are desirable to eliminate intersection conflicts. Where grade separations are not feasible, assignment of right of way by traffic signals should be considered. Where traffic is not heavy, "STOP" or "YIELD" signs for either the path or the cross street (depending on volumes) may suffice.

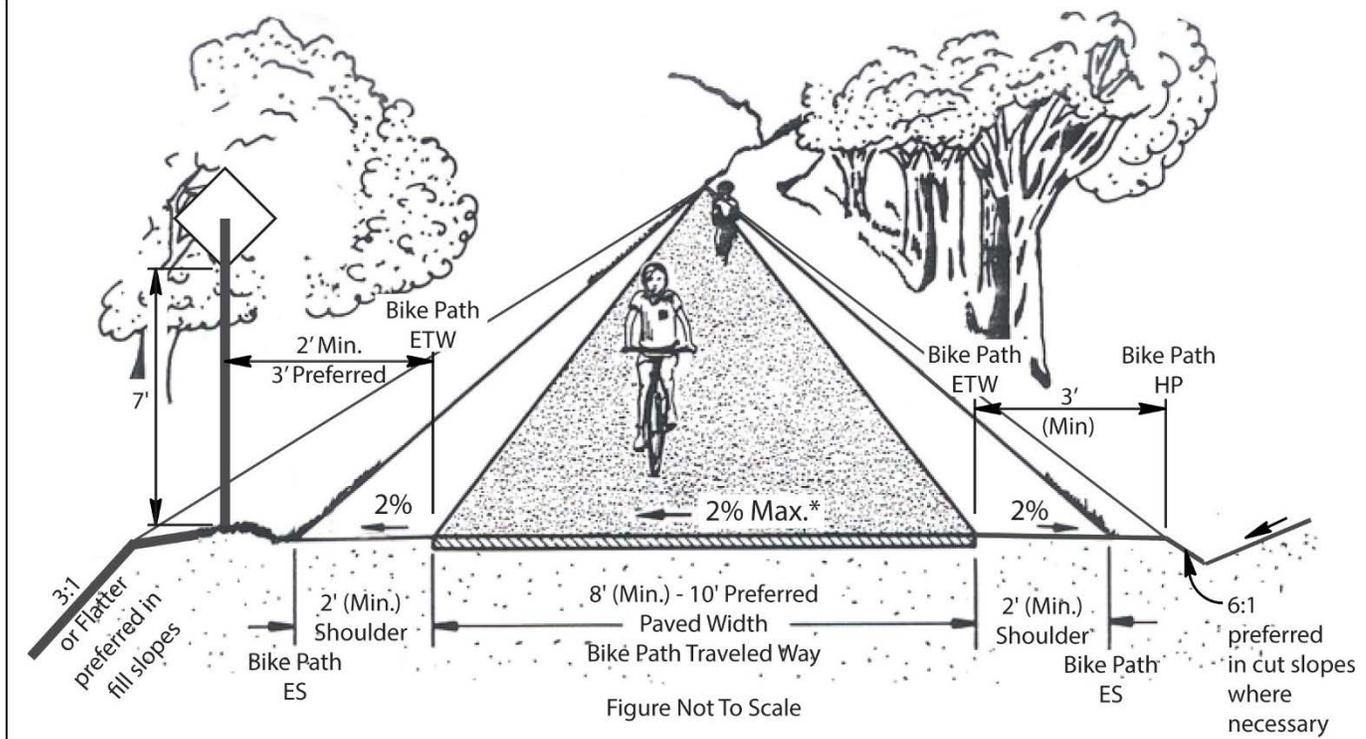
Bicycle path intersections and their approaches should be on relatively flat grades. Stopping sight distances at intersections should be checked and adequate warning should be given to permit bicyclists to stop before reaching the intersection, especially on downgrades. When contemplating the placement of signs the designer is to discuss the proposed sign details with their Traffic Liaison so that conflicts may be minimized. Bicycle versus motor vehicle collisions may occur more often at intersections, where bicyclists misuse pedestrian crosswalks; thus, this should be avoided.

When crossing an arterial street, the crossing should either occur at the pedestrian crossing, where vehicles can be expected to stop, or at a location completely out of the influence of any intersection to permit adequate opportunity for bicyclists to see turning vehicles. When crossing at midblock locations, right of way should be assigned by devices such as "YIELD" signs, "STOP" signs, or traffic signals which can be activated by bicyclists. Even when crossing within or adjacent to the pedestrian crossing, "STOP" or "YIELD" signs for bicyclists should be placed to minimize potential for conflict resulting from turning autos. Where bike path "STOP" or "YIELD" signs are visible to approaching motor vehicle traffic, they should be shielded to avoid confusion. In some cases, Bike Xing signs may be placed in advance of the crossing to alert motorists. Ramps should be installed in the curbs, to preserve the utility of the bike path. Ramps should be the same width as the bicycle paths. Curb cuts and ramps should provide a smooth transition between the bicycle paths and the roadway.

Assignment of rights of way is necessary where bicycle paths intersect roadways or other bicycle paths. See the California MUTCD, Section 9B.03 and Figure 9B-7 for guidance on signals and signs for rights of way assignment at bicycle path intersections.

- (6) *Paving at Crossings.* At unpaved roadway or driveway crossings, including bike paths or pedestrian walkways, the crossing roadway or driveway shall be paved a minimum of 15 feet to minimize or eliminate gravel intrusion on the path. The pavement structure at the crossing should be adequate to sustain the expected loading at that location
- (7) *Bike Paths Parallel and Adjacent to Streets and Highways.* A wide separation is recommended between bike paths and adjacent highways (see Figure 1003.1B). **The minimum separation between the edge of pavement of a one-way or a two-way bicycle path and the edge of traveled way of a parallel road or street shall be 5 feet plus the standard shoulder widths. Bike paths within the clear recovery zone of freeways shall**

Figure 1003.1A
Two-Way Class I Bikeway (Bike Path)

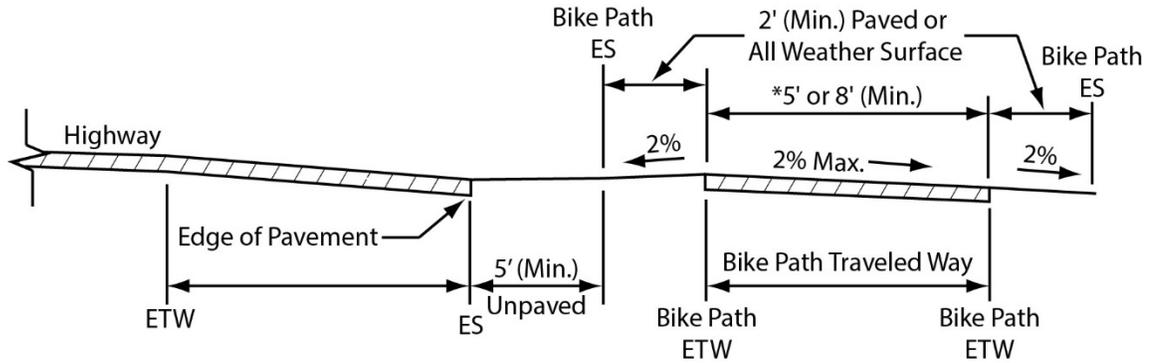


NOTES:

- (1) See Index 1003.1(13) for pavement structure guidance of bike path.
- (2) For sign clearances, see California MUTCD, Figure 9B-1.
- * 1% cross-slope minimum.

Figure 1003.1B

Typical Cross Section of Class I Bikeway (Bike Path) Parallel to Highway



NOTE:

(1) See Index 1003.1(6) for guidance on separation between bike paths and highways.

* One-Way: 5' Minimum Width

Two-Way: 8' Minimum Width

include a physical barrier separation. The separation is unpaved and does not include curbs or sidewalks. Separations less than 10 feet from the edge of the shoulder are to include landscaping or other features that provide a continuous barrier to prevent bicyclists from encroaching onto the highway. Suitable barriers may include fences or dense shrubs if design speeds are less than or equal to 45 miles per hour. Obstacles low to the ground or intermittent obstacles (e.g., curbs, dikes, raised traffic bars, posts connected by cable or wire, flexible channelizers, etc.) are not to be used because bicyclists could fall over these obstacles and into the roadway.

Bike paths immediately adjacent to streets and highways are not recommended. While they can provide separation between vehicles and nonmotorized traffic, they typically introduce significant conflicts at intersections. In addition, they can create conflicts with passengers at public transit facilities, and with vehicle occupants crossing the path. They are not a substitute for designing the road to meet bicyclist's mobility needs. Use of bicycle paths adjacent to roads is not mandatory in California, and many bicyclists will perceive these paths as offering a lower level of mobility compared with traveling on the road, particularly for utility trips. Careful consideration regarding how to address the above points needs to be weighed against the perceived benefits of providing a bike path adjacent to a street or highway. Factors such as urban density, the number of conflict points, the presence or absence of a sidewalk, speed and volume should be considered.

(8) *Bike Paths in the Median of Highway or Roadway.* Bike paths should not be placed in the median of a State highway or local road, and shall not be in the median of a freeway or expressway. Bike paths in the median are generally not recommended because they may require movements contrary to normal rules of the road. Specific problems with such facilities may include:

(a) Right-turns by bicyclists from the median of roadways are unexpected by motorists.

- (b) Devoting separate phases to bicyclist movements to and from a median path at signalized intersections increases intersection delay.
- (c) Left-turning motorists must cross one direction of motor vehicle traffic and two directions of bicycle traffic, which may increase conflicts.
- (d) Where intersections are infrequent, bicyclists may choose to enter or exit bike paths at midblock.
- (e) Where medians are landscaped, visibility between bicyclists on the path and motorists at intersections may be diminished. See Chapter 900 for planting guidance.

(9) *Bicycle Path Design Speed.* The design speed of bicycle paths is established using the same principles as those applied to highway design speeds. **The design speed given in Table 1003.1 shall be the minimum.**

**Table 1003.1
Bike Path Design Speeds**

Type of Facility	Design Speed (mph) ⁽¹⁾
Bike Paths with Mopeds Prohibited	20
Bike Paths with Mopeds Permitted	30
Bike Paths on Long Downgrades (steeper than 4%, and longer than 500')	30

NOTE:

- (1) On bike paths with mopeds prohibited, a lower design speed can be used for the crest vertical curve, equivalent to 1 mile per hour per percent grade for grades exceeding a vertical rise of 10 feet, when at a crest in path.

Installation of "speed bumps", gates, obstacles, posts, fences or other similar features intended to cause bicyclists to slow down are not to be used.

- (10) *Horizontal Alignment and Superelevation.* The minimum radius of curvature negotiable by a bicycle is a function of the superelevation of the bicycle path surface, the coefficient of friction between the bicycle tires and the bicycle path surface, and the speed of the bicycle.

For all bicycle path applications the maximum superelevation rate is 2 percent.

The minimum radius of curvature should be 90 feet for 20 miles per hour, 160 feet for 25 mile per hour and 260 feet for 30 miles per hour. No superelevation is needed for radius of curvature meeting or exceeding 100 feet for 20 miles per hour, 180 feet for 25 miles per hour, and 320 feet for 30 miles per hour. When curve radii smaller than those given because of right of way, topographical or other considerations, standard curve warning signs and supplemental pavement markings should be installed. The negative effects of nonstandard curves can also be partially offset by widening the pavement through the curves.

- (11) *Stopping Sight Distance.* To provide bicyclists with an opportunity to see and react to the unexpected, a bicycle path should be designed with adequate stopping sight distances. **The minimum stopping sight distance based on design speed shall be 125 feet for 20 miles per hour, 175 feet for 25 miles per hour and 230 feet for 30 miles per hour.** The distance required to bring a bicycle to a full controlled stop is a function of the bicyclist's perception and brake reaction time, the initial speed of the bicycle, the coefficient of friction between the tires and the pavement, and the braking ability of the bicycle.

Stopping sight distance is measured from a bicyclist's eyes, which are assumed to be 4 ½ feet above the pavement surface to an object ½-foot high on the pavement surface.

- (12) *Length of Crest Vertical Curves.* Figure 1003.1C indicates the minimum lengths of crest vertical curves for varying design speeds.
- (13) *Lateral Clearance on Horizontal Curves.* Figure 1003.1D indicates the minimum clearances to line of sight obstructions, *m*, for horizontal curves. It is assumed that the

bicyclist's eyes are 4 ½ feet above the pavement surface to an object ½-foot high on the pavement surface.

Bicyclists frequently ride abreast of each other on bicycle paths, and on narrow bicycle paths, bicyclists have a tendency to ride near the middle of the path. For these reasons, lateral clearances on horizontal curves should be calculated based on the sum of the stopping sight distances for bicyclists traveling in opposite directions around the curve. Where this is not possible or feasible, the following or combination thereof should be provided: (a) the path through the curve should be widened to a minimum paved width of 14 feet; and (b) a yellow center line curve warning sign and advisory speed limit signs should be installed.

- (14) *Grades.* Bike path grades must meet DIB 82. The maximum grade rate recommended for bike paths should be 5 percent. Sustained grades should be limited to 2 percent.
- (15) *Pavement Structure.* The pavement material and structure of a bike path should be designed in the same manner as a highway, with a recommendation from the District Materials Branch. It is important to construct and maintain a smooth, well drained, all-weather riding surface with skid resistant qualities, free of vegetation growth. Principal loads will normally be from maintenance and emergency vehicles.
- (16) *Drainage.* For proper drainage, the surface of a bike path should have a minimum cross slope of 1 percent to reduce ponding and a maximum of 2 percent per DIB 82. Sloping of the traveled way in one direction usually simplifies longitudinal drainage design and surface construction, and accordingly is the preferred practice. **The bike path shoulder shall slope away from the traveled way at 2 percent to 5 percent to reduce ponding and minimize debris from flowing onto the bike path.** Ordinarily, surface drainage from the path will be adequately dissipated as it flows down the gently sloping shoulder. However, when a bike path is constructed on the side of a hill, a drainage ditch of suitable dimensions may be necessary on the uphill side to intercept the hillside drainage. Where necessary, catch

basins with drains should be provided to carry intercepted water under the path. Such ditches should be designed in such a way that no undue obstacle is presented to bicyclists.

Culverts or bridges are necessary where a bike path crosses a drainage channel.

(17) *Entry Control for Bicycle Paths.* Obstacle posts and gates are fixed objects and placement within the bicycle path traveled way can cause them to be an obstruction to bicyclists. Obstacles such as posts or gates may be considered only when other measures have failed to stop unauthorized motor vehicle entry. Also, these obstacles may be considered only where safety and other issues posed by actual unauthorized vehicle entry are more serious than the safety and access issues posed to bicyclists, pedestrians and other authorized path users by the obstacles.

The 3-step approach to prevent unauthorized vehicle entry is:

- (a) Post signs identifying the entry as a bicycle path with regulatory signs prohibiting motor vehicle entry where roads and bicycle paths cross and at other path entry points.
- (b) Design the path entry so it does not look like a vehicle access and makes intentional access by unauthorized users more difficult. Dividing a path into two one-way paths prior to the intersection, separated by low plantings or other features not conducive to motor vehicle use, can discourage motorists from entering and reduce driver error.
- (c) Assess whether signing and path entry design prevents or minimizes unauthorized entry to tolerable levels. If there are documented issues caused by unauthorized motor vehicle entry, and other methods have proven ineffective, assess whether the issues posed by unauthorized vehicle entry exceed the crash risks and access issues posed by obstacles.

If the decision is made to add bollards, plantings or similar obstacles, they should be:

- Yielding to minimize injury to bicyclists and pedestrians who may strike them.

- Removable or moveable (such as gates) for emergency and maintenance access must leave a flush surface when removed.
- Reflectorized for nighttime visibility and painted, coated, or manufactured of material in a bright color to enhanced daytime visibility.
- Illuminated when necessary.
- Spaced to leave a minimum of 5 feet of clearance of paved area between obstacles (measured from face of obstacle to face of adjacent obstacle). Symmetrically about the center line of the path.
- Positioned so an even number of bicycle travel lanes are created, with a minimum of two paths of travel. An odd number of openings increase the risk of head-on collisions if traffic in both directions tries to use the same opening.
- Placed so additional, non-centerline/lane line posts are located a minimum of 2 feet from the edge of pavement.
- Delineated as shown in California MUTCD Figure 9C-2.
- Provide special advance warning signs or painted pavement markings if sight distance is limited.
- Placed 10 to 30 feet back from an intersection, and 5 to 10 feet from a bridge, so bicyclists approach the obstacle straight-on and maintenance vehicles can pull off the road.
- Placed beyond the clear zone on the crossing highway, otherwise breakaway.

When physical obstacles are needed to control unauthorized vehicle access, a single non-removable, flexible, post on the path centerline with a separate gate for emergency/maintenance vehicle access next to the path, is preferred. The gate should swinging away from the path,

Fold-down obstacle posts or bollards shall not be used within the paved area of bicycle paths. They are often left in the folded down

Figure 1003.1C

**Minimum Length of Bicycle Path Crest Vertical Curve (L)
Based on Stopping Sight Distance (S)**

$$L = 2S - \frac{1600}{A} \quad \text{when } S > L$$

Double line represents $S = L$

L = Minimum length of vertical curve – feet

A = Algebraic grade difference - %

$$L = \frac{AS^2}{1600} \quad \text{when } S < L$$

S = Stopping sight distance – feet

Refer to Index 1003.1(11) to determine “S”, for a given design speed “V”

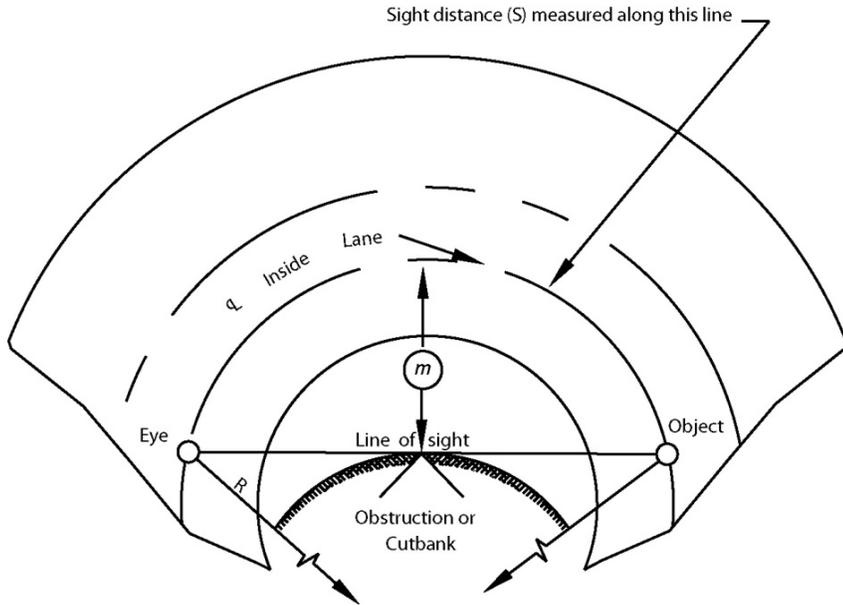
Height of cyclist eye = 4½ feet

Height of object = ½ foot

A (%)	S = Stopping Sight Distance (ft)													
	70	90	110	125	130	150	170	175	190	210	230	250	270	
3													7	
4										20	60	100	140	
5							20	30	60	100	140	180	220	
6		S > L				33	73	83	113	153	193	233	270	
7				21	31	71	111	121	151	191	231	273	319	
8			20	50	60	100	140	150	180	221	265	313	365	
9		2	42	72	82	122	162	172	203	248	298	352	410	
10		20	60	90	100	140	181	191	226	276	331	391	456	
11		35	75	105	115	155	199	211	248	303	364	430	501	
12	7	47	87	117	127	169	217	230	271	331	397	469	547	
13	17	57	97	127	137	183	235	249	293	358	430	508	592	S < L
14	26	66	106	137	148	197	253	268	316	386	463	547	638	
15	33	73	113	146	158	211	271	287	338	413	496	586	683	
16	40	80	121	156	169	225	289	306	361	441	529	625	729	
17	46	86	129	166	180	239	307	325	384	469	562	664	775	

Figure 1003.1D

Minimum Lateral Clearance (m) on Bicycle Path Horizontal Curves



S = Sight distance in feet.

R = Radius of \mathcal{C} of lane in feet.

m = Distance from \mathcal{C} of lane in feet.

Refer to Index 1003.1(11) to determine "S" for a given design speed "V".

Angle is expressed in degrees

$$m = R \left[1 - \cos \left(\frac{28.655}{R} \right) \right]$$

$$S = \frac{R}{28.655} \left[\cos^{-1} \left(\frac{R-m}{R} \right) \right]$$

Formula applies only when S is equal to or less than length of curve.

Line of sight is 28" above \mathcal{C} inside lane at point of obstruction.

Height of bicyclist's eye is 4 1/2 feet.

R (ft)	S = Stopping Sight Distance (ft)										
	60	80	100	120	140	160	180	200	220	240	260
25	15.9										
50	8.7	15.2	23.0	31.9	41.5						
75	5.9	10.4	16.1	22.8	30.4	38.8	47.8	57.4	67.2		
95	4.7	8.3	12.9	18.3	24.7	31.8	39.5	48.0	56.9	66.3	75.9
125		6.3	9.9	14.1	19.1	24.7	31.0	37.9	45.4	53.3	1.76
155		5.1	8.0	11.5	15.5	20.2	25.4	31.2	37.4	44.2	51.4
175		4.6	7.1	10.2	13.8	18.0	22.6	27.8	33.5	39.6	46.1
200		4.0	6.2	8.9	12.1	15.8	19.9	24.5	29.5	34.9	40.8
225			5.5	8.0	10.8	14.1	17.8	21.9	26.4	31.3	36.5
250			5.0	7.2	9.7	12.7	16.0	19.7	23.8	28.3	33.1
275			4.5	6.5	8.9	11.6	14.6	18.0	21.7	25.8	30.2
300			4.2	6.0	8.1	10.6	13.4	16.5	19.9	23.7	27.7
350				5.1	7.0	9.1	11.5	14.2	17.1	20.4	23.9
390				4.6	6.3	8.2	10.3	12.8	15.4	18.3	21.5
500					4.9	6.4	8.1	10.0	12.1	14.3	16.8
565					4.3	5.7	7.2	8.8	10.7	12.7	14.9
600					4.1	5.3	6.7	8.3	10.1	12.0	14.0
700						4.6	5.8	7.1	8.6	10.3	12.0
800						4.0	5.1	6.2	7.6	9.0	10.5
900							4.5	5.6	6.7	8.0	9.4
1000							4.0	5.0	6.0	7.2	8.4

position, which presents a crash hazard to bicyclists and pedestrians. When vehicles drive across fold-down obstacles, they can be broken from their hinges, leaving twisted and jagged obstructions that project a few inches from the path surface.

Obstacle posts or gates must not be used to force bicyclists to slow down, stop or dismount. Treatments used to reduce vehicle speeds may be used where it is desirable to reduce bicycle speeds.

For obstacle post visibility marking, and pavement markings, see the California MUTCD, Section 9C.101(CA).

- (18) *Lighting.* Fixed-source lighting raises awareness of conflicts along paths and at intersections. In addition, lighting allows the bicyclist to see the bicycle path direction, surface conditions, and obstacles. Lighting for bicycle paths is important and should be considered where nighttime use is not prohibited, in sag curves (see Index 201.5), at intersections, at locations where nighttime security could be a problem, and where obstacles deter unauthorized vehicle entry to bicycle paths. See Index 1003.1(17). Daytime lighting should also be considered through underpasses or tunnels.

Depending on the location, average maintained horizontal illumination levels of 5 lux to 22 lux should be considered. Where special security problems exist, higher illumination levels may be considered. Light standards (poles) should meet the recommended horizontal and vertical clearances. Luminaires and standards should be at a scale appropriate for a pedestrian or bicycle path. For additional guidance on lighting, consult with the District Traffic Electrical Unit.

1003.2 Class II Bikeways (Bike Lanes)

Design guidance that address the safety and mobility needs of bicyclists on Class II bikeways (bike lanes) is distributed throughout this manual where appropriate.

For Class II bikeway signing and lane markings, see the California MUTCD, Section 9C.04.

1003.3 Class III Bikeways (Bike Routes)

Class III bikeways (bike routes) are intended to provide continuity to the bikeway system. Bike routes are established along through routes not served by Class I or II bikeways, or to connect discontinuous segments of bikeway (normally bike lanes). Class III facilities are facilities shared with motor vehicles on the street, which are established by placing bike route signs along roadways. Additional enhancement of Class III facilities can be provided by adding shared roadway markings along the route. For application and placement of signs and pavement markings, see the California MUTCD Section 9C.

Minimum widths for Class III bikeways are represented, in the minimum standards for highway lanes and shoulder.

Since bicyclists are permitted on all highways (except prohibited freeways), the decision to designate the route as a bikeway should be based on the advisability of encouraging bicycle travel on the route and other factors listed below.

- (1) *On-street Bike Route Criteria.* To be of benefit to bicyclists, bike routes should offer a higher degree of service than alternative streets. Routes should be signed only if some of the following apply:
- (a) They provide for through and direct travel in bicycle-demand corridors.
 - (b) Connect discontinuous segments of bike lanes.
 - (c) They provide traffic actuated signals for bicycles and appropriate assignment of right of way at intersections to give greater priority to bicyclists, as compared with alternative streets.
 - (d) Street parking has been removed or restricted in areas of critical width to provide improved safety.
 - (e) Surface imperfections or irregularities have been corrected (e.g., utility covers adjusted to grade, potholes filled, etc.).
 - (f) Maintenance of the route will be at a higher standard than that of other comparable streets (e.g., more frequent street sweeping).

(2) *Sidewalk as Bikeway.* Sidewalks are not to be designated for bicycle travel. Wide sidewalks that do not meet design standards for bicycle paths or bicycle routes also may not meet the safety and mobility needs of bicyclists. Wide sidewalks can encourage higher speed bicycle use and can increase the potential for conflicts with turning traffic at intersections as well as with pedestrians and fixed objects.

In residential areas, sidewalk riding by young children too inexperienced to ride in the street is common. It is inappropriate to sign these facilities as bikeways because it may lead bicyclists to think it is designed to meet their safety and mobility needs. Bicyclists should not be encouraged (through signing) to ride their bicycles on facilities that are not designed to accommodate bicycle travel.

(3) *Shared Transit and Bikeways.* Transit lanes and bicycles are generally not compatible, and present risks to bicyclists. Therefore sharing exclusive use transit lanes for buses with bicycles is discouraged.

Bus and bicycle lane sharing should be considered only under special circumstances to provide bikeway continuity, such as:

- (a) If bus operating speed is 25 miles per hour or below.
- (b) If the grade of the facility is 5 percent or less.

1003.4 Trails

Trails are generally, unpaved multipurpose facilities suitable for recreational use by hikers, pedestrians, equestrians, and off-road bicyclists. While many Class I facilities are named as trails (e.g. Iron Horse Regional Trail, San Gabriel River Trail), trails as defined here do not meet Class I bikeways standards and should not be signed as bicycle paths. Where equestrians are expected, a separate equestrian trail should be provided. See DIB 82 for trail requirements for ADA. See Index 208.7 for equestrian undercrossing guidance.

- Pavement requirements for bicycle travel are not suitable for horses. Horses require softer surfaces to avoid leg injuries.

- Bicyclists may not be aware of the need to go slow or of the separation need when approaching or passing a horse. Horses reacting to perceived danger from predators may behave unpredictably; thus, if a bicyclist appears suddenly within their visual field, especially from behind they may bolt. To help horses not be surprised by a bicyclist, good visibility should be provided at all points on equestrian paths.
- When a corridor includes equestrian paths and Class I bikeways, the widest possible lateral separation should be provided between the two. A physical obstacle, such as an open rail fence, adjacent to the equestrian trail may be beneficial to induce horses to shy away from the bikeway, as long as the obstacle does not block visibility between the equestrian trail and bicycle path.

See FHWA-EP-01-027, *Designing Sidewalks and Trails for Access* and DIB 82 for additional design guidance.

1003.5 Miscellaneous Criteria

The following are miscellaneous bicycle treatment criteria. Specific application to Class I, and III bikeways are noted. Criteria that are not noted as applying only to bikeways apply to any highway, roadways and shoulders, except freeways where bicycles are prohibited), without regard to whether or not bikeways are established.

Bicycle Paths on Bridges – See Topic 208.

- (1) *Pavement Surface Quality.* The surface to be used by bicyclists should be smooth, free of potholes, and with uniform pavement edges.
- (2) *Drainage Grates, Manhole Covers, and Driveways.* Drainage inlet grates, manhole covers, etc., should be located out of the travel path of bicyclists whenever possible. When such items are in an area that may be used for bicycle travel, they shall be designed and installed in a manner that meets bicycle surface requirements. See Standard Plans. They shall be maintained flush with the surface when resurfacing.

If grate inlets are to be located in roadway or shoulder areas (except freeways where bicycles

are prohibited) the inlet design guidance of Index 837.2(2) applies.

Future driveway construction should avoid construction of a vertical lip from the driveway to the gutter, as the lip may create a problem for bicyclists when entering from the edge of the roadway at a flat angle. If a lip is deemed necessary, the height should be limited to ½ inch.

- (3) *At-grade Railroad Crossings and Cattle Guards.* Whenever it is necessary for a Class I bikeway, highway or roadway to cross railroad tracks, special care must be taken to ensure that the safety of users is protected. The crossing must be at least as wide as the traveled way of the facility. Wherever possible, the crossing should be straight and at right angles to the rails. For bikeways or highways that cross tracks and where a skew is unavoidable, the shoulder or bikeway should be widened, to permit bicyclists to cross at right angles (see Figure 1003.5). If this is not possible, special construction and materials should be considered to keep the flangeway depth and width to a minimum.

Pavement should be maintained so ridge buildup does not occur next to the rails. In some cases, timber plank crossings can be justified and can provide for a smoother crossing.

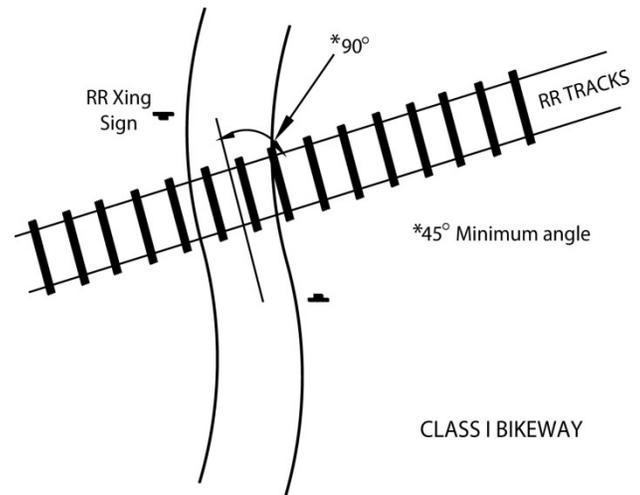
All railroad crossings are regulated by the California Public Utilities Commission (CPUC). All new bicycle path railroad crossings must be approved by the CPUC. Necessary railroad protection will be determined based on a joint field review involving the applicant, the railroad company, and the CPUC.

Cattle guards across any roadway are to be clearly marked with adequate advance warning. Cattle guards are only to be used where there is no other alternative to manage livestock.

The California MUTCD has specific guidance on Rail and Light Rail crossings. See Part 8 of the California MUTCD.

Figure 1003.5

**Railroad Crossing
Class I Bikeway**



NOTE:

See Index 403.3 Angle of Intersection for Class II and Class III facilities.