FOREWORD

Purpose

This manual was prepared for the California Department of Transportation (Department) by the Division of Design for use on the California State highway system. This manual establishes uniform policies and procedures to carry out the State highway design functions of the Department. It is neither intended as, nor does it establish, a legal standard for these functions.

The standards, procedures, and requirements established and discussed herein are for the information and guidance of the officers and employees of the Department.

Many of the instructions given herein are subject to amendment as conditions and experience warrant. Special situations may call for deviation from policies and procedures, subject to Division of Design approval, or such other approval as may be specifically provided for in the text of this manual.

It is not intended that any standard of conduct or duty toward the public shall be created or imposed by the publication of this manual. Statements as to the duties and responsibilities of any given classification of officers or employees mentioned herein refer solely to duties or responsibilities owed by these in such classification to their superiors. However, in their official contacts, each employee should recognize the necessity for good relations with the public.

Scope

This manual is not a textbook or a substitute for engineering knowledge, experience, or judgment. It includes techniques as well as graphs and tables not ordinarily found in textbooks. These are intended as aids in the quick solutions of field and office problems. Except for new developments, no attempt is made to detail basic engineering techniques; for these, standard textbooks should be used.

Form

The loose-leaf form was chosen because it facilitates change and expansion. New instructions or updates will be issued as sheets in the format of this manual and made available on-line on the Department Design website: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm. The new instructions or updates may consist of additional sheets or new sheets to be substituted for those superseded. Users of this manual are encouraged to utilize the most recent version available on-line on the Department Design website.

Organization of the Manual

A decimal numbering system is used which permits identification by chapter, topic, and index, each of which is a subdivision of the preceding classification. For example:

Chapter 40 Federal-Aid

Topic 42 Federal-Aid System

Index 42.2 Interstate

The upper corner of each page shows the page number and the date of issue.

Use the Table of Contents

The Table of Contents gives the index number and page number for each topical paragraph together with corresponding dates of issue. If the holder of the manual chooses to maintain a paper copy, the holder is responsible for keeping the paper copy up to date and current. Revised Table of Contents will be issued on the Department Design website as the need arises.

Use of the English and Metric Editions of the Highway Design Manual

This Sixth Edition of the Highway Design Manual is in U.S. Customary (English) units. Departmental policy established by Director's Policy 15-R1 and Deputy Directive Number 12-R1, both effective October 2006, state that the Department has adopted the use of the U.S. Customary (English) units as its preferred system of units and measures. All projects designed and constructed in English units shall follow the standards in this manual.

The Metric standards contained in the Fifth Edition of the Highway Design Manual, and related publications, are to continue to be used if the specific project was granted an exception to continue to be delivered in Metric units. Only those projects identified, approved, and disclosed per Project Delivery Directive 3 (PID-03) are allowed to continue to be advertised and constructed on the State highway system using Metric units.

Use of the HDM as a Reference in Other Media

No warranty is made regarding the results of use of this Caltrans Highway Design Manual (HDM) or that the HDM will accurately and reliably test construction designs for compliance with any Federal, State or industry standards, or that the HDM will predict or test the safety or other feature or a structure. Engineering judgment must be used to apply the HDM to designs and to adjust designs to fit individual site conditions. The HDM is not intended to be a substitute for engineering knowledge, experience or judgment. In no event shall the Departmentbe liable for costs of procurement of substitute goods, loss of profits, or for any indirect, special, consequential or incidental damages, however caused, by use of the HDM. The Department shall not be liable for any claims in connection with the use of the HDM, including without limitation, liability arising from third-party claims, liability related to the quality of calculations or the safety or quality or structures, liability for scheduling delays or re-design, retrofit or re-work of structures, or other similar liability.

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)
Celcius 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*as in j*i*g, *a* as in *a*-bout)
1 000 000 = 10^6	mega	M	as in *mega*-phone
1000 = 10^3	kilo	k	kill' oh
100 = 10^2	*hecto	h	heck' toe
10 = 10^1	*deko	da	deck' a (*a* as in *a*-bout)
0.1 = 10^{-1}	*deci	d	as in *deci*-mal
0.01 = 10^{-2}	*centi	c	as in *centi*-pede
0.001 = 10^{-3}	milli	m	as in *mili*-tary
0.000 001 = 10^{-6}	micro	μ	as in *micro*-phone
0.000 000 001 = 10^{-9}	nano	n	nan' oh (*an* as in *ant*)

* 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
	acre ft	1233.49	m^3
Mass	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
	short ton	0.9072	tonne (1000 kg)
Density	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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CHAPTER 10
DIVISION OF DESIGN

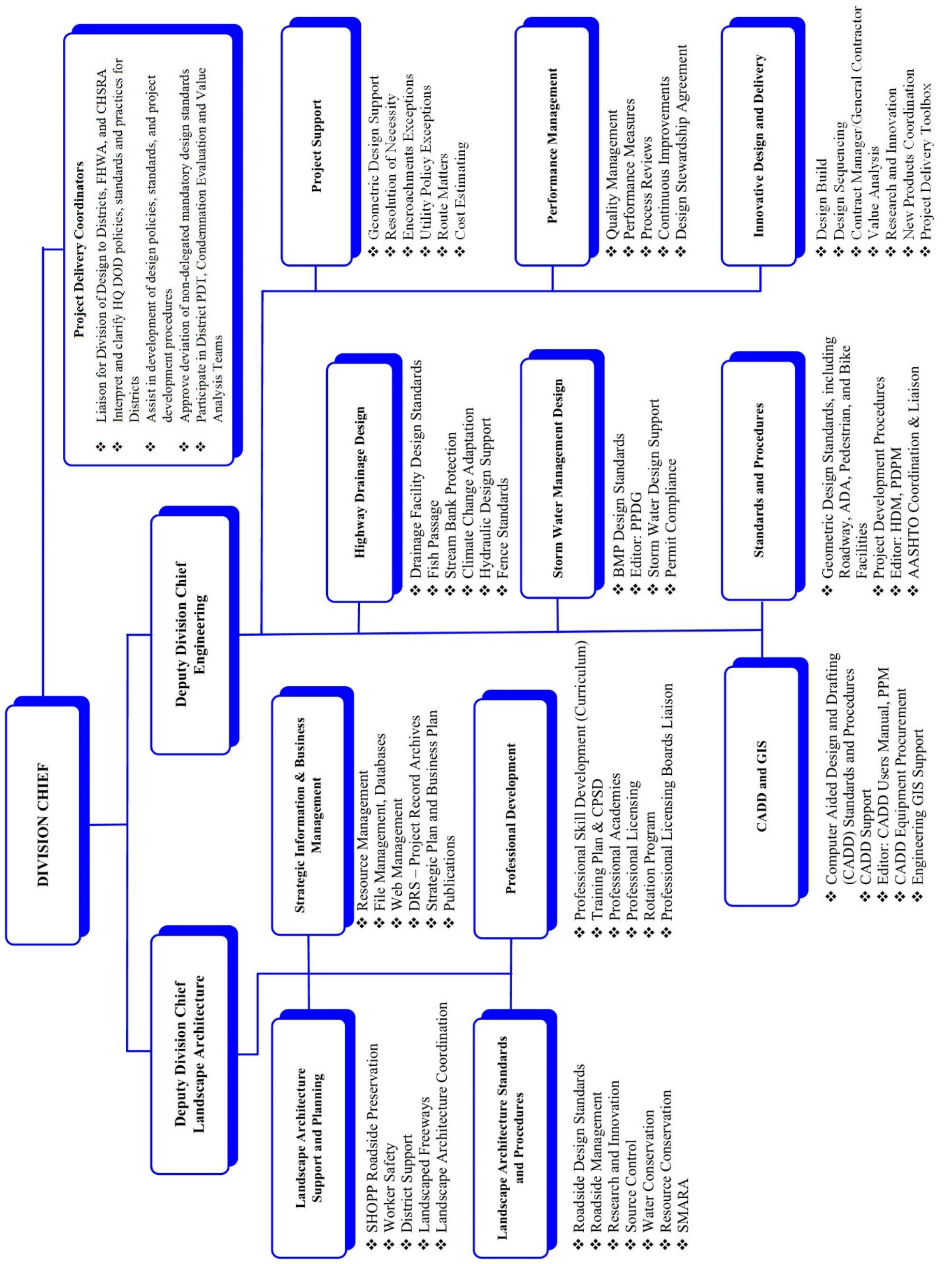
Topic 11 - Organization and Functions

Index 11.1 - Organization

The Division of Design (DOD), a part of Project Delivery, is comprised of the Engineering Program with the following offices: CADD and GIS, Highway Drainage Design, Innovative Design and Delivery, Performance Management, Project Support, Standards and Procedures, Storm Water Management Design; as well as the Landscape Architecture Program with the following offices: Landscape Architecture Standards and Procedures, Landscape Architecture Support and Planning, Professional Development, and Strategic Information & Business Management. Additionally, the Project Delivery Coordinators represent the Chief, DOD, in the California Department of Transportation (Department) Districts, maintaining liaison and coordinating District and Headquarters activities, ensuring consistent and uniform application of statewide policies, standards, procedures, guidelines and practices. See Figure 11.1 for information on the functional duties performed by the various offices in the DOD.

As the Chief Design Engineer within the DOD, the Chief, Division of Design provides technical and procedural advice and assistance to the Districts in support of the development of transportation projects as follows: establishes, maintains and monitors the project development process in accord with all applicable State and Federal laws and regulations; establishes engineering standards and procedures for application of standards on a statewide basis; approves exceptions to non-delegated mandatory design standards; monitors project development related reports, facilitates performance management and process improvement activities. The Chief, DOD also is a member of the AASHTO Subcommittee on Design.

Figure 11.1
Division of Design Functional Organization Chart



CHAPTER 20 DESIGNATION OF HIGHWAY ROUTES

Topic 21 - Highway Route Numbers

Index 21.1 - Legislative Route Numbers and Descriptions

The Legislature designates all State highway routes and assigns route numbers. The description and number of each route are contained in Chapter 2, Article 3 of the Streets and Highways Code. These route numbers are used for all administrative purposes.

The Legislature has stated its intent that the routes of the State Highway System serve the State's heavily traveled rural and urban corridors, that they connect the communities and regions of the State, and that they serve the State's economy by connecting centers of commerce, industry, agriculture, mineral wealth, and recreation.

A legislative route description generally runs south to north or west to east. To the extent possible, the number used on each route's guide signs is the same as the legislatively designated route number.

A specific location on any State highway is described by its post mile designation (formerly known as kilometer post). Post miles typically start at the west or south county line and end at the east or north county line. Generally, post mile information is available in the Caltrans State Highway Log, and is maintained by the Department's, Office of System Management Planning.

21.2 Sign Route Numbers

Each route in the State Highway System is given a unique number for identification and signed with distinctive numbered Interstate, U.S. or California State route shields to guide public travel. Route numbers used on one system are not duplicated on another system. Odd numbered routes are generally south to north and even numbered routes are generally west to east.

(1) *Interstate and Defense Highways (Interstate System).* The Interstate System is a network of freeways of national importance, created by Congress and constructed with Federal-aid Interstate System funds. Routes in the system are signed with the Interstate route shields (See Index 42.2 and Figure 21.1) and the general numbering convention is as follows: routes with one or two-digit numbers are north-south or east-west through routes, routes with three-digit numbers, the first of which is odd, are interstate spur routes. For example, I-110 is a spur route off of I-10. Routes in three-digit numbers, the first of which is even, are loops through or belt routes around cities. I-805 in San Diego is an example of a loop off of I-5. The numbering of Interstate routes was developed by AASHTO with concurrence by the states.

    Renumbering of Interstate routes requires the approval of AASHTO to assure conformity with established numbering procedures. Such revisions also are a system action that must be approved by the Federal Highway Administrator.

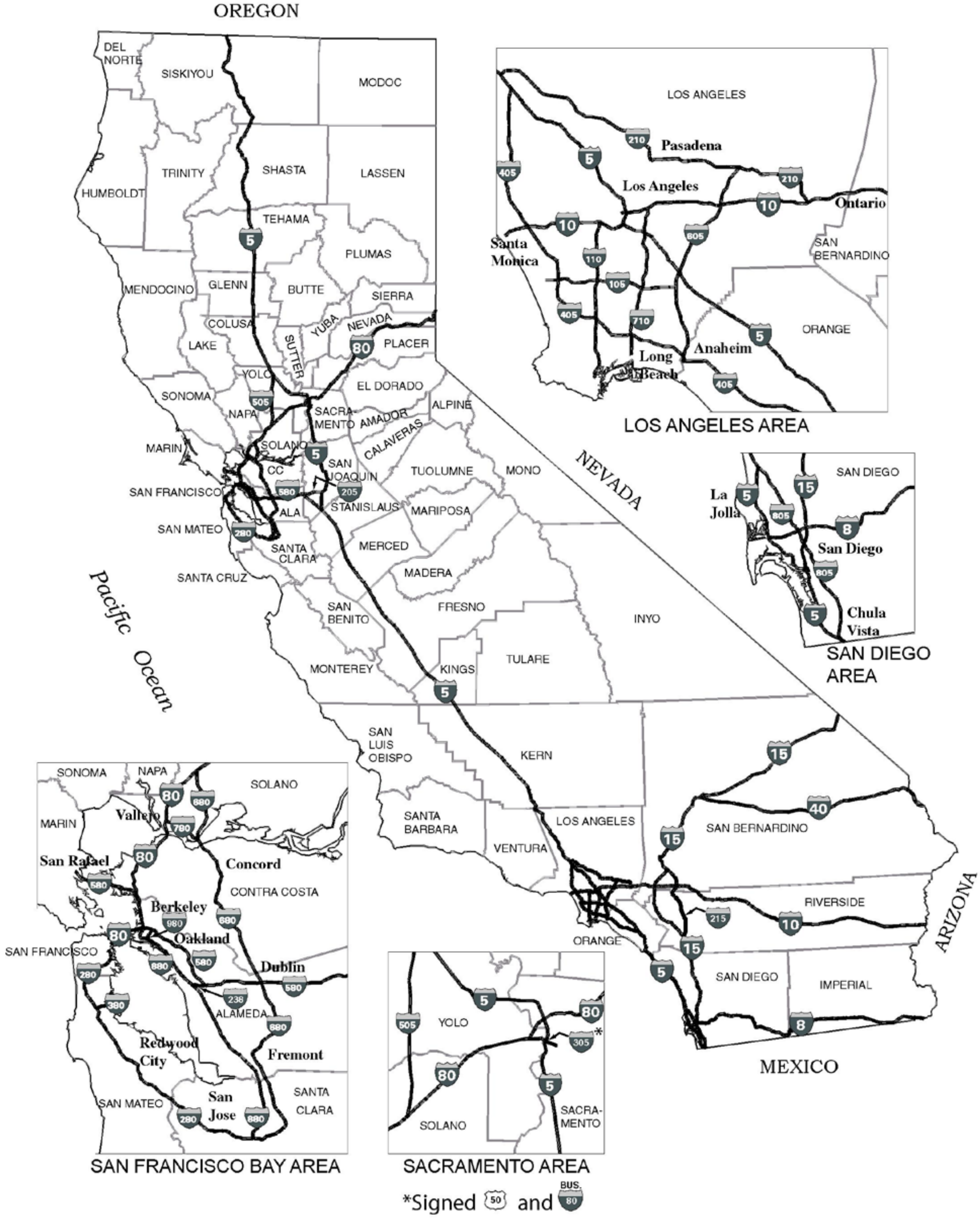
    The Transportation System Information Program is responsible for processing requests for changes to the system to AASHTO and FHWA for their consideration.

(2) *United States Numbered Routes.* United States Numbered Routes are a network of State highways of statewide and national importance. These highways can be conventional roadways or freeways.

    The establishment of a U.S. number as a guide for interstate travel over certain roads has no connection with Federal control, any Federal-aid System, or Federal construction financing. The Executive Committee of AASHTO, with the concurrence of the states, has full authority for numbering U.S. routes.

    The Transportation System Information Program is responsible for processing requests for numbering U.S. routes to AASHTO for their consideration.

Figure 21.1 Interstate Highway System in California



(3) *State Sign Routes.* State Sign Routes are State highways within the State, other than the above signed routes, which are distinctively signed to serve intrastate and interstate travel.

(4) *Business Routes.* A Business Route generally is a local street or road in a city or urban area, designated by the same route number as the through Interstate, U.S., or State highway to which it is connected, with the words “Business Route" attached to the identifying route shields. The Business Route designation provides guidance for the traveling public to leave the main highway at one end of a city or urban area, patronize local businesses, and continue on to rejoin the main route at the opposite end of the city or urban area.

The Transportation System Information Program is responsible for approval of Business Route designations. Applications for Business Route designation and signing must be made by written request from the local government agency to the Chief of the Transportation System Information Program. U.S. and Interstate Business Routes require approval by the AASHTO Executive Committee.

CHAPTER 40
FEDERAL-AID

Topic 41 - Enabling Legislation

Index 41.1 - General

The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 is the first transportation legislation since the Interstate System was enacted. ISTEA has changed the established Federal-Aid system. During the 20 years prior to ISTEA there were four Federal-Aid systems: Interstate, Primary, Secondary, and Urban. Now, instead of four Federal-aid systems there are two, the National Highway System (NHS) and the Interstate System, which is a component of the National Highway System.

In 2005, the Safe, Accountable, Flexible, Efficient Transportation Enhancement Act, Legacy for the Users, better known as SAFETEA-LU, was passed. SAFETEA-LU, invests in highway, transit and safety programs. While ISTEA created new federal-aid programs, SAFETEA-LU continued those programs such as the Surface Transportation Program, National Highway System, Congestion Mitigation and Air Quality Improvement Program and the Bridge Replacement and Rehabilitation Program.

A variety of other programs also continued to exist to provide flexibility in determining transportation solutions and promote a multi-modal system approach. Some of these programs include those that target funding for rail and transit projects while others provide funds for environmental enhancement such as habitat mitigation and wetland banking. Numerous other funding categories are also available for use during the six year term of the act.

Topic 42 - Federal-Aid System

42.1 National Highway System

After consultation with the States, in 1995 the Secretary of Transportation proposed a National Highway System (NHS) consisting of approximately 160,000 miles across the United States. The NHS consists of all Interstate routes, a large percentage of urban and rural principal arterials, the defense strategic highway network, and strategic highway connectors.

42.2 Interstate

As a result of ISTEA the Interstate System is a part of the NHS, but will retain its separate identity and receive separate funding. SAFETEA-LU continued those funding programs for the Interstate and NHS; however, SAFETEA-LU concentrated on safety and congestion. SAFETEA-LU also addressed other important aspects of an effective and efficient highway program.

Topic 43 - Federal-Aid Programs

43.1 Surface Transportation Program (STP)

The Surface Transportation Program is a funding program which may be used for roads (including NHS) that are not functionally classified as local or rural minor collectors. These roads are now collectively referred to as Federal-aid roads.

The STP includes safety and enhancement programs. Ten percent of the STP funds must be used for safety construction activities, hazard elimination and rail-highway crossings. Another ten percent of the program is designated for transportation enhancement, which encompasses a broad range of environmental related activities. The remainder of the STP funds are divided as follows; 50 percent is to be divided between areas of the State based on population; the remaining 30 percent can be used in any area.

43.2 California Stewardship and Oversight Agreement with FHWA

The goal under the Stewardship and Oversight Agreement (Agreement) is to document the roles and responsibilities of the FHWA's California Division Office and Caltrans with respect to project approvals and related responsibilities, and to document the methods of oversight which will be used to efficiently and effectively deliver the Federal-aid Highway Program. The Agreement states that "Caltrans [Department] and the FHWA will jointly determine which projects are considered to be projects of Division or Corporate Interest (PODI and/or POCI). The initial PODI and POCI determination will be made at the Caltrans [Department] District level in conjunction with the

FHWA." Projects not selected as PODIs or POCIs will be considered as Delegated Projects and, the Department will have approval authority for all aspects of a Federal-aid project, except those which may not be delegated by federal law (requiring FHWA approval). For the Delegated Projects, FHWA will verify compliance with federal regulations via annual program and process reviews. See the Project Development Procedures Manual for other essential procedures regarding the Stewardship and Oversight Agreement between the Department and FHWA. For additional information see the FHWA webpage on Stewardship and Oversight. See the Department Design website for the current Stewardship and Oversight Agreement between FHWA California Division Office and Caltrans.

43.3 Congestion Mitigation and Air Quality Improvement Program (CMAQ)

The Congestion Mitigation and Air Quality Improvement Program directs funds toward transportation projects in Clean Air Act non-attainment areas for ozone and carbon monoxide. Projects using CMAQ funds contribute to meeting the attainment of national ambient area air quality standards. CMAQ funds may not be used for projects which will increase capacity for single occupant vehicles. Exceptions might include HOV lanes which allow single occupant vehicles at other than peak travel times or auxiliary lanes.

43.4 Bridge Replacement and Rehabilitation Program

The Bridge Replacement and Rehabilitation Program was continued in order to provide assistance for any bridge on public roads. Caltrans, Division of Engineering Services, Office of Structures Maintenance and Investigation, develops the bridge sufficiency rating for bridges on the State system and sets a sufficiency threshold for the use of Bridge Replacement and Rehabilitation Funds.

43.5 Federal Lands Program

The Federal Lands Program authorizations are available through three categories: Indian Reservation roads, Parkways and Park roads, and Public Lands Highways (which incorporates the previous Forest Highway category).

43.6 Highway Safety Improvement Program

SAFETEA-LU established the Highway Safety Improvement Program (HSIP) as a core Federal-aid program for safety funding to achieve a significant reduction in traffic fatalities and serious injuries on all public roads. The state apportionment of funds is subject to a set aside for construction and operational improvements on high risk rural roads (HRR). HRR are functionally classified as rural major or minor collectors or rural roads with a fatal or injury crash rate above statewide average for those functional classes of roadways, injury crash rates above those functional classes of roadways, or those roads which are likely to experience an increase in traffic volumes that could lead to a crash rate in excess of the statewide rate.

The HSIP also created a planning process for safety which is overseen by the Department. The Strategic Highway Safety Plan is developed with input from stakeholders to better coordinate funding and safety efforts on the State highway system

43.7 Special Programs

Special Program funds are allocated for projects which generally fall into the following groups: Special Projects-High Cost Bridge, Congestion Relief, High Priority Corridors on the NHS, Rural and Urban Access, Priority Intermodal and Innovative Projects; National High Speed Ground Transportation Programs; Scenic Byways Program; Use of Safety Belts and Motorcycle Helmets; National Recreational Trails Program; Emergency Relief.

Topic 44 - Funding Determination

44.1 Funding Eligibility

Each Federal program has certain criteria and requirements. During design the project engineer is to consult with the FHWA reviewer to determine the appropriate Federal program each individual project is eligible for and the level of future Federal involvement. The final determination to request Federal participation will be made by Caltrans, Budgets Program, Federal Resource Branch.

44.2 Federal Participation Ratio

SAFETEA-LU designates the percentage of Federal participation in several programs and fund types. The Interstate System reimbursement allotment is approximately 90 percent. The remainder of projects on the NHS, STP and CMAQ reimbursement allotments is approximately 80 percent. For certain safety improvements, the federal share may be up to 100%. FHWA determines the final detailed ratio based on a formula applied to each State. Contact Caltrans, Budgets Program, Federal Resources Branch for the most current reimbursement rates.

44.3 Emergency Relief

Emergency opening projects are funded 100 percent for the first 180 days following a disaster. For restoration projects and emergency opening projects after 180 days Federal participation is pro-rated.

CHAPTER 60
NOMENCLATURE

Unless indicated otherwise in this manual, wherever the following abbreviations, terms, or phrases are used, their intent and meaning shall be as identified in this Chapter.

Topic 61 - Abbreviations

Index 61.1 - Official Names

AASHTO	American Association of State Highway and Transportation Officials
Caltrans or Department	California Department of Transportation
CFR	Code of Federal Regulations
CTC or Commission	California Transportation Commission
DES	Division of Engineering Services
District	Department of Transportation Districts
DOT	U.S. Department of Transportation
DOD	Division of Design
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
GS	Geotechnical Services
METS	Office of Materials Engineering and Testing Services
OAP	Office of Asphalt Pavement
OCPPF	Office of Concrete Pavement and Pavement Foundations
PP	Pavement Program
PS&E	Plans, Specifications, and Estimate
PUC	Public Utilities Commission
SD	Structure Design
SHOPP	State Highway Operation and Protection Plan
STIP	State Transportation Improvement Program

Topic 62 - Definitions

62.1 Geometric Cross Section

(1) *Lane.*

(a) Auxiliary Lane--The portion of the roadway for weaving, truck climbing, speed change, or for other purposes supplementary to through movement.

(b) Lane Numbering--On a multilane roadway, the lanes available for through travel in the same direction are numbered from left to right when facing in the direction of travel.

(c) Multiple Lanes--Freeways and conventional highways are sometimes defined by the number of through lanes in both directions. Thus an 8-lane freeway has 4 through lanes in each direction. Likewise, a 4-lane conventional highway has 2 through lanes in each direction. Lanes that are not equally distributed to each direction would otherwise be described as appropriate.

(d) Median Lane--A speed change lane within the median to accommodate left turning vehicles.

(e) Speed Change Lane--An auxiliary lane, including tapered areas, primarily for the acceleration or deceleration of vehicles when entering or leaving the through lanes.

(f) Traffic Lane/Vehicle Lane--The portion of the traveled way for the movement of a single line of vehicles, both motor vehicle and bicycle.

(2) *Bikeways.*

(a) Class I Bikeway (Bike Path). Provides a completely separated facility for the exclusive use of bicycles and pedestrians with crossflow by vehicles minimized.

(b) Class II Bikeway (Bike Lane). Provides a striped lane for one-way bike travel on a street or highway.

(c) Class III Bikeway (Bike Route). Provides for shared use with pedestrian or motor vehicle traffic.

(d) Class IV Bikeway (Separated Bikeway). Provides for the exclusive use of bicycles and includes a separation (e.g., grade separation, flexible posts, inflexible physical barrier, or on-street parking) required between the separated bikeway and the through vehicular traffic.

(3) *Maintenance Vehicle Pullout (MVP).* Paved areas, or appropriate all weather surfaces, adjacent to the shoulder for field personnel to

park off the traveled way and access the work site.

(4) *Median.* The portion of a divided highway separating the traveled ways in opposite directions.

(5) *Outer Separation.* The portion of an arterial highway between the traveled ways of a roadway and a frontage street or road.

(6) *Roadbed.* That portion of the roadway extending from curb line to curb line or shoulder line to shoulder line. Divided highways are considered to have two roadbeds.

(7) *Roadside.* A general term denoting the area adjoining the outer edge of the roadbed to the right of way line. Extensive areas between the roadbeds of a divided highway may also be considered roadside.

(8) *Roadway.* That portion of the highway included between the outside lines of the sidewalks, or curbs and gutters, or side ditches including also the appertaining structures, and all slopes, ditches, channels, waterways, and other features necessary for proper drainage and protection.

(9) *Shoulder.* The portion of the roadway contiguous with the traveled way for the accommodation of stopped vehicles, for emergency use, for errant vehicle recovery, and for lateral support of base and surface courses. The shoulder may accommodate on-street parking as well as bicyclists and pedestrians, see the guidance in this manual as well as DIB 82.

(10) *Sidewalk.* A surfaced pedestrian way contiguous to a roadbed used by the public where the need for which is created primarily by the local land use. See DIB 82 for further guidance.

(11) *Traveled Way*. The portion of the roadway for the movement of vehicles and bicycles, exclusive of shoulders.

62.2 Highway Structures

(1) *Illustration of Types of Structures.* Figure 62.2 illustrates the names given to common types of structures used in highway construction. This nomenclature must be used in all phases of planning.

(2) *Bridges.* A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads; and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of (buried) arches, or extreme ends of openings for (buried) multiple boxes. It may also include (buried) multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

(3) *Culverts.* A type of buried structure without a bridge number, see Index 806.2.

Any structure that fits the definition of a bridge shall be assigned a bridge number by Structure Maintenance and Investigation. Buried structures that meet the definition of a bridge but are made of a collection of culverts will only be considered as bridges for the purposes of design and structural maintenance record, not for definitions in specifications.

Buried structures, with or without bridge numbers, covered by Caltrans Standard Plans can be designed by the District. Culvert modifications to Standard Plans can be designed by the District and shall be reviewed by the Division of Engineering Services. Buried structure with a bridge number but not covered by Standard Plans shall be designed by the Division of Engineering Services.

62.3 Highway Types

(1) *Freeway.* A freeway, as defined by statute, is a highway in respect to which the owners of abutting lands have no right or easement of access to or from their abutting lands or in respect to which such owners have only limited or restricted right or easement of access. This statutory definition also includes expressways.

The engineering definitions for use in this manual are:

(a) Freeway--A divided arterial highway with full control of access and with grade separations at intersections.

(b) Expressway--An arterial highway with at least partial control of access, which may or may not be divided or have grade separations at intersections.

(2) *Controlled Access Highway.* In situations where it has been determined advisable by the Director or the CTC, a facility may be designated a "controlled access highway" in lieu of the designation "freeway". All statutory provisions pertaining to freeways and expressways apply to controlled access highways.

(3) *Conventional Highway.* A highway without control of access which may or may not be divided. Grade separations at intersections or access control may be used when justified at spot locations.

(4) *Highway.* In general a public right of way for the purpose of travel or transportation.

(a) Alley--A road passing through a continuous row of houses, buildings, etc. that permits access from the local street network to backyards, garages, etc.

(b) Arterial Highway--A general term denoting a highway primarily for through travel usually on a continuous route.

(c) Bypass--An arterial highway that permits users to avoid part or all of a city or town center, a suburban area, or an urban area.

(d) Collector-Distributor Road--A separated freeway system adjacent to a freeway, which connects two or more local road ramps or freeway connections to the freeway at a limited number of points.

(e) Collector Road--A route that serves travel of primarily intracounty rather than statewide importance in rural areas or a route that serves both land access and traffic circulation within a residential neighborhood, as well as commercial and industrial areas in urban and suburban areas.

(f) Divided Highway--A highway with separated roadbeds for traffic traveling in opposing directions.

(g) Major Street or Major Highway--An arterial highway with intersections at grade and direct access to abutting property on which geometric design and traffic control measures are used to expedite the safe movement of through traffic.

(h) Through Street or Through Highway--The highway or portion thereof at the entrance to which vehicular traffic from intersecting highways is regulated by "STOP" signs or traffic control signals or is controlled when entering on a separate right-turn roadway by a "YIELD" sign.

(5) *Parkway.* An arterial highway for non-commercial vehicles, with full or partial control of access, which is typically located within a park or a ribbon of park-like development.

(6) *Scenic Highway.* A State or county highway, in total or in part, that is recognized for its scenic value, protected by a locally adopted corridor protection program, and has been officially designated by the Department.

(7) *Street or Road.*

(a) Cul-de-Sac Street--A local street open at one end only, with special provisions for turning around.

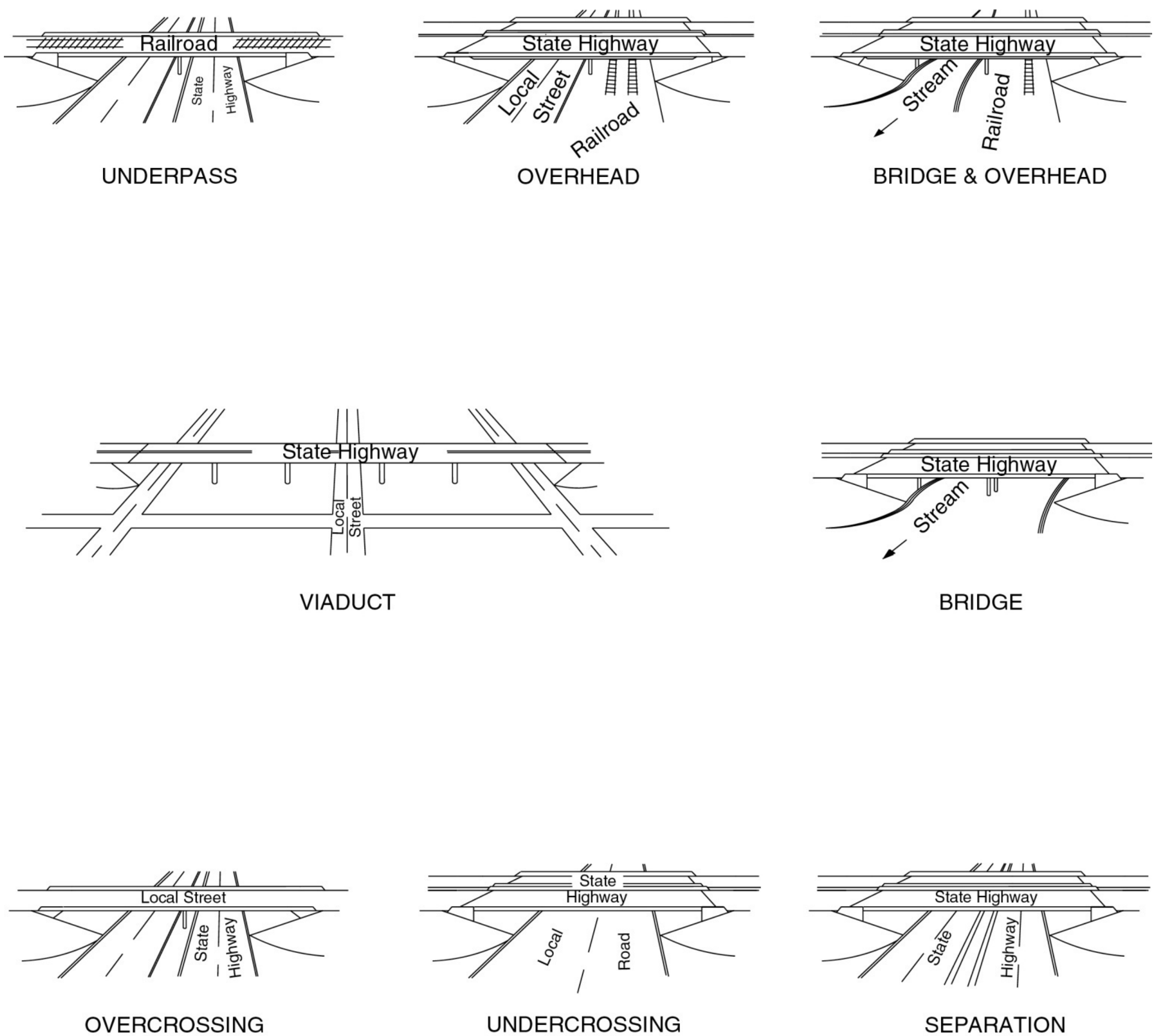
(b) Dead End Street/No Outlet--A local street open at one end only, without special provisions for turning around.

(c) Frontage Street or Road--A local street or road auxiliary to and located on the side of an arterial highway for service to abutting property and adjacent areas and for control of access.

(d) Local Street or Local Road--A street or road primarily for access to residence, business or other abutting property.

(e) Private Road or Private Driveway--A way or place in private ownership and used for travel by the owner and those having express or implied permission from the

Figure 62.2
Types of Structures



owner but not by other members of the public.

(f) Street--A way or place that is publicly maintained and open for the use of the public to travel. Street includes highway.

(g) Toll Road, Bridge or Tunnel--A highway, bridge, or tunnel open to traffic only upon payment of a toll or fee.

(8) *Throughway.* A conventional highway or a suburban arterial in developed or developing areas, that is characterized by lower density (not built out) land uses, adjacent undeveloped land or parkland, direct access to abutting property, at-grade intersections, and that may have shoulders with or without curb and gutter.

62.4 Interchanges and Intersections at Grade

(1) *Central Island.* The raised area in the center of a roundabout around which traffic circulates. The central island does not necessarily need to be circular in shape.

(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.

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

(6) 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.

(7) *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.

(8) *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.

(9) *Local Topsoil.* Existing soil obtained from the "A" and "O" soil horizons within the project limits, typically during excavation activities.

(10) *"O" Soil Horizon*. The surface layer consisting of loose and partly decaying organic matter.

(11) *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.

(12) *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.

(13) *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.

(14) *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.

(15) *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.

(16) *Street Furniture*. Features such as newspaper boxes, bicycle racks, bus shelters, benches, art or drinking fountains that occupy space on or alongside pedestrian sidewalks.

(17) *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 design elements for a particular section of highway or bike path.

(b) Operating Speed--The speed at which drivers are observed operating their vehicles during free-flow conditions. The 85th percentile of the distribution of a representative sample of observed speeds is used most frequently to measure the operating speed associated with a particular location or geometric feature.

(c) Posted Speed--The speed limit determined by law and shown on the speed limit sign.

(d) High Speed – A speed greater than 45 mph.

(e) Low Speed – A speed less than or equal to 45 mph.

(f) Running Speed--The speed over a specified section of highway, being the distance divided by running time. The average for all traffic, or component thereof, is the summation of distances divided by the summation of running times.

(14) Traffic. A general term used throughout this manual referring to the passage of people, vehicles and/or bicycles along a transportation route.

(15) Traffic Control Devices.

(a) Markings--All pavement and curb markings, object markers, delineators, colored pavements, barricades, channelizing devices, and islands used to convey regulations, guidance, or warning to users.

(b) Sign--Any traffic control device that is intended to communicate specific information to users through a word, symbol and/or arrow legend. Signs do not include highway traffic signals or pavement markings, delineators, or channelizing devices.

(c) Highway Traffic Signal--A power-operated control device by which traffic is warned or directed to take a specific action. These devices do not include signals at toll plazas, power-operated signs, illuminated pavement markers, warning lights, or steady burning electrical lamps.

(d) Changeable Message Sign--An electronic traffic sign used on roadways to give travelers information about traffic congestion, accidents, roadwork zones, speed limits or any dynamic information about current driving conditions.

(16) Volume. The number of vehicles passing a given point during a specified period of time.

(17) Weaving. The crossing of traffic streams moving in the same general direction accomplished by merging and diverging.

(18) Ramp Metering. A vehicular traffic management strategy which utilizes a system of traffic signals on freeway entrance and connector ramps to regulate the volume of vehicles entering a freeway corridor in order to maximize the efficiency of the freeway and thereby minimizing the total delay in the transportation corridor.

62.9 Drainage

See Chapter 800 for definition of drainage terms.

62.10 Users

(1) Bicycle. A device propelled via chain, belt or gears, exclusively by human power.

(2) Bus. Any vehicle owned or operated by a publicly owned or operated transit system, or operated under contract with a publicly owned or operated transit system, and used to provide to the general public, regularly scheduled transportation for which a fare is charged. A general public paratransit vehicle is not a transit bus.

(3) *Bus Rapid Transit (BRT).* A flexible rubber-tired rapid-transit mode that combines stations, vehicles, services, exclusive running ways, and Intelligent Transportation System elements into an integrated system with a strong positive identity that evokes a unique image.

(4) *Commuter Rail.* Traditional rapid and heavy rail passenger service intended to provide travel options in suburban and urban areas. Corridor lengths are typically shorter than intercity passenger rail services. Top operating speeds are in the range of 90 to 110 miles per hour. The tracks may or may not be shared with freight trains and typically are in a separate right of way.

(5) *Conventional Rail.* Traditional intercity passenger rail and interregional freight rail. Top operating speeds are in the range of 60 to 110 miles per hour. The tracks may or may not be shared by passenger and freight trains and typically run within their own right of way corridor.

(6) *Design Vehicle*. The largest vehicle commonly expected on a particular roadway. Descriptions of these vehicles are found in Index 404.4.

(7) *Equestrian.* A rider on horseback.

(8) *High Speed Rail.* A type of intercity and interregional passenger rail service that operates significantly faster than conventional rail. Top operating speeds are typically 150 to 220 miles per hour. These trains may be powered by overhead high voltage lines or technologies such as Maglev. The tracks are grade separated within a separate controlled access right of way and may or may not be shared with freight trains.

(9) *Light Rail.* A form of urban transit that uses rail cars on fixed rails in a right of way that may or may not be grade separated. Motorized vehicles and bicycles may share the same transportation corridor. These railcars are typically electrically driven with power supplied from an overhead line rather than an electrified third rail. Top operating speeds are typically 60 miles per hour.

(10) *Pedestrian*. A person who is afoot or who is using any of the following: (a) a means of conveyance propelled by human power other than a bicycle, or (b) an electric personal assistive mobility device. Includes a person who is operating a self-propelled wheelchair, motorized tricycle, or motorized quadricycle and, by reason of physical disability, is otherwise unable to move about as a pedestrian as specified in part (a) above.

(11) *Street Car, Trams or Trolley.* A passenger rail vehicle which runs on tracks along public urban streets and also sometimes on separate rights of way. It may also run between cities and/or towns, and/or partially grade separated structures.

(12) *Transit.* Includes light rail; commuter rail; motorbus; street car, tram, trolley bus; BRT; automated guideway; and demand responsive vehicles. The most common application is for motorbus transit. See Index 404.4 for a description of the design vehicle as related to buses.

(13) *Vehicle.* A device to move, propel or draw a person upon a highway, except a device on rails or propelled exclusively by human power. This definition, abstracted from the CVC, is intended to refer to motor vehicles, excluding those devices necessary to provide mobility to persons with disabilities.

CHAPTER 80 APPLICATION OF DESIGN STANDARDS

Topic 81 - Project Development Overview

Index 81.1 - Philosophy

The Project Development process seeks to provide a degree of mobility to users of the transportation system that is in balance with other values. In the development of transportation projects, social, economic, and environmental effects must be considered fully along with technical issues so that final decisions are made in the best overall public interest. Attention should be given to such considerations as:

(a) Need to provide transportation for all users (motorists, bicyclists, transit riders, and pedestrians) of the facility and transportation modes.

(b) Attainment of community goals and objectives.

(c) Needs of low mobility and disadvantaged groups.

(d) Costs and benefits of eliminating or minimizing adverse effects on natural resources, environmental values, public services, aesthetic values, and community and individual integrity .

(e) Planning based on realistic financial estimates.

(f) The cost, ease, and safety of maintaining whatever is built.

Proper consideration of these items requires that a facility be viewed from the perspectives of the user, the nearby community, and larger statewide interests. For the user, efficient travel, mode selection, and safety are paramount concerns. At the same time, the community often is more concerned about local aesthetic, social, and economic impacts. The general population, however, tends to be interested in how successfully a project functions as part of the overall transportation system and how large a share of available capital resources it consumes. Therefore, individual projects must be selected for construction on the basis of overall system benefits as well as community goals, plans, and values.

Decisions must also emphasize the connectivity between the different transportation modes so that they work together effectively.

The goal is to increase person and goods throughput, highway mobility and safety in a manner that is compatible with, or which enhances, adjacent community values and plans.

81.2 Highway Context

The context of a highway is a critical factor when developing the purpose and need statement for a project in addition to making fundamental design decisions such as its typical cross section and when selecting the design elements and aesthetic features such as street furniture and construction materials. Designing a highway that is sensitive to, and respectful of, the surrounding context is critical for project success in the minds of the Department and our stakeholders.

A "one-size-fits-all" design philosophy is not Departmental policy. Designers need to be aware of and sensitive to land use, community context and the associated user needs of the facility. In some instances, the design criteria and standards in this manual are based on the land use contexts in which the State highway is located, for instance: large population areas and downtowns in urban areas, small rural towns and communities, suburban commercial/residential areas, and rural corridors. This approach ensures the standards are flexible, and the approach allows and encourages methods to minimize impacts on scenic, historic, archaeological, environmental, and other important resources.

Beyond their intended transportation benefits, State highways can significantly impact the civic, social and economic conditions of local communities. Designing transportation facilities that integrate the local transportation and land uses while making the design responsive to the other needs of the community support the livability of the community and are usually a complementary goal to meeting the transportation needs of the users of the State highway system.

To do this successfully, the designer needs to have an understanding of the area surrounding the highway and the users of the highway, its function within the regional and State transportation systems, (which includes all transportation modes), and the level of access control needed. To gain this understanding, the designer must consult the Transportation Concept Reports and work with the planning division and the local agencies.

In this manual, the following concepts are used to discuss the context of a highway:

- Place Type - the surrounding built and natural environment;
- Type of Highway - the role the highway plays in terms of providing regional or interregional connectivity and local access; and,
- Access Control - the degree of connection or separation between the highway and the surrounding land use.

A "Main Street" design is not specific to a certain place type, but is a design philosophy to be applied on State highways that also function as community streets. A "Main Street" design serves pedestrians, bicyclists, businesses and public transit with motorized traffic operating at speeds of 20 to 40 miles per hour. See the Department's "Main Street, California" document for more information.

81.3 Place Types

A place type describes the area's physical environment and the land uses surrounding the State highway. The place types described below are intentionally broad. Place types should be agreed upon in partnership with all of the project stakeholders; however, there likely may be more than one place type within the limits of a project. Ultimately, the place types selected can be used to determine the appropriate application of the guidance provided in this manual. These place type definitions are independent of the Federal government definitions of urban and rural areas. See Title 23 United States Code, Section 13 for further information.

Identifying the appropriate place type(s) involves discussions with the project sponsors, ideally through the Project Development Team (PDT) process, and requires coordination with the land use planning activities associated with the on-going local and regional planning activities. Extensive community engagement throughout both the project planning and project development processes helps to formulate context sensitive project alternatives and transportation facilities that coordinate with the local land uses.

The following place types are used in this manual:

(1) Rural Areas. Rural areas are typically sparsely settled and developed. They can consist of protected federal and State lands, agricultural lands, and may include tourist and recreational destinations. However, as rural lands transition into rural communities, they can become more developed and suburban and urban-like by providing for a mixture of housing, commercial, industrial and public institutions. For the use of this manual, rural areas have been subcategorized as Natural Corridors, Developing Corridors and City/Town Centers (Rural Main Streets).

(a) Natural Corridors. Typically, the desire in these corridors is to preserve the natural and scenic countryside while at the same time provide transportation services to support the travel and tourism that occurs when visiting these locations. Examples of this place type are: National/State Forests and Parklands; agricultural lands with scattered farm buildings and residences; and, low density development. See Topic 109 for additional information.

(b) Developing Corridors. State highways traveling through these lands tend to be increasingly clustered with industrial, commercial, and residential areas as they lead into a rural city or town center. These corridors can be a transition zone among the aforementioned areas. Highways associated with these locations help to deliver tourists, but they also need to support the local communities and their local economies. In addition, these highways also serve a role and should be efficient at moving people and goods between regions.

Industrial, commercial and retail buildings tend to be located separately from housing and are typically set back from the highway with parking areas placed in front. Truck traffic on these highways tends to serve the needs of these industrial, commercial and retail buildings; however, there will be a component of the truck traffic that is transporting their loads inter-regionally. Therefore, corridors in areas that are in transition may need to accommodate design vehicles.

(c) City or Town Centers (Rural Main Streets). State highways in this scenario are usually a conventional main street through the rural city or town, or they may be the only main street. The use of the State highway in this environment varies depending upon the individual community, as does the mix of buildings, services, businesses, and public spaces. Transit is often present and should be incorporated into the transportation system as appropriate. Transportation improvement projects on these main street highways can be more complicated and costly than similar projects in more rural settings. A balance usually needs to be maintained between the needs of the through traffic and those of the local main street environment. Thus, analyzing the pedestrian and bicyclist needs early in the development of the project and then following through on the agreements during the design of highway projects in these locations can be especially important. Accommodating the pedestrian and bicyclist needs concurrently in projects leads to greater efficiency in the use of funding.

(2) Suburban Areas. Suburban areas lead into and can completely surround urban areas. A mixture of land uses is typical in suburban areas. This land use mixture can consist of housing, retail businesses and services, and may include regional centers such as shopping malls and other similar regional destinations; which are usually associated with suburban communities (cities and towns) that can be connected with larger urban centers and cities. Assessing the needs of pedestrians, bicyclists, and transit users in concert with the vehicular needs of motorists and truck drivers is necessary during the project planning, development and design of highway projects in these locations. Accommodating all of these needs concurrently into a project leads to greater efficiency in the use of funding. For the use of this manual, suburban areas have been categorized as either Lower Density/Residential Neighborhoods or Higher Density/Regional Community Centers (Suburban Main Streets).

(a) Lower Density / Residential Neighborhoods. State highways typically do not cross through this place type. This place type usually feeds users onto the State highway system and is typically under the jurisdiction of a local entity. State highways, if they do interact with this place type, usually just connect at the edges of them where the pedestrians, bicyclists, and motor vehicle operators integrate into the highway system that includes transit facilities.

(b) Higher Density / Regional Community Centers (Suburban Main Streets). As suburban areas grow they tend to merge together into each other's boundaries. Growth in some locations can create "Megacommunities." While these megacommunities seem to function as individual cities, they typically have multiple distinct community centers that require highways with the capacity to serve not only each center, but the center-to-center traveler needs. These areas typically require the State highway to serve not only the originally urbanized area, but also the newer suburban areas that have been created where the housing, shopping and employment opportunities are all centered. Anticipating and accommodating growth in this place type can be a challenge. State and local governments, the business community and citizens groups, and metropolitan planning organizations all need to agree on how to meet the community needs, and at times the interregional needs of the highway.

(3) Urban and Urbanized Areas. Urban areas generally are the major population centers in the State. Large numbers of people live in

these urbanized areas where growth is expected to continue. Bicycling, transit, and walking are important transportation modes in these areas and as the facilities for pedestrians, transit and bicyclists expand in these areas, the percentage and number of travelers walking, using transit and bicycling is also likely to increase. State agencies and the local governmental entities, the business community and citizens groups, congestion management agencies and the local/regional metropolitan planning organization (MPO) need to all agree upon the concept of the transportation facilities being provided so that the community needs can be met.

Urban areas are typically high-density locations such as central business districts, downtown communities, and major activity centers. They have a full range of land uses and are associated with a large diversity of activities. For the use of place types in this manual, urban areas have been categorized as Lower Density Parklands and Residential Neighborhoods and Higher Density Urban Main Streets. Higher Density Urban Main Streets have been further characterized as Community Centers and Downtown Cores.

(a) Lower Density Parklands and Residential Neighborhoods. Large numbers of people live in these urbanized areas and bicycling, transit and walking are important transportation modes in these areas. Parklands can enhance these neighborhoods and parkland preservation is a concern, as well as, access to support travel and tourism to the parklands.

(b) High Density Urban Main Streets.

- Community Centers or Corridor. Strategically improving the design and function of the existing State highways that cross these centers is typically a concern. Providing transportation options to enhancing these urban neighborhoods that combine highway, transit, passenger rail, walking, and biking options are desirable, while they also help promote tourism and shopping.
- Downtown Cores. Similar to community centers, much of the transportation system has already been built and its footprint in the community needs to be preserved while its use may need to be reallocated. Successfully meeting the mobility needs of a major metropolitan downtown core area requires a balanced approach. Such an approach is typically used to enhance the existing transportation network's performance by adding capacity to the highways, sidewalks, and transit stations for all of the users of the system, and/or adding such enhancement features as HOV lanes, BRT, walkable corridors, etc. Right of way is limited and costly to purchase in these locations. Delivery truck traffic that supports the downtown core businesses can also create problems.

The HEPGIS tool on the FHWA website is available to determine if the project is in an urban area. Urban areas are found on the Highway Information tab of the tool.

81.4 Type of Highway

Much of the following terminology is either already discussed in Chapter 20 or defined in Topic 62. The additional information in this portion of the manual is being provided to connect these terms with the guidance that is being provided.

(1) Functional Classification. One of the first steps in the highway design process is to define the function that the facility is to serve. The two major considerations in functionally classifying a highway are access and throughput. Access and mobility are inversely related; as access is increased, mobility decreases. In the AASHTO "A Policy on Geometric Design of Highways and Streets", highways are functionally classified first as either urban or rural. The hierarchy of the functional highway system within either an urban or rural area consists of the following:

- Principal arterial - main movement (high mobility, limited access) Typically 4 lanes or more;
- Minor arterial - interconnects principal arterials (moderate mobility, limited access) Typically 2 or 3 lanes with turn lanes to benefit through traffic;
- Collectors - connects local roads to arterials (moderate mobility, moderate access) with few businesses; and,
- Local roads and streets - permits access to abutting land (high access, limited mobility).

The California Road System (CRS) maps are the official functional classification maps approved by Federal highway Administration. These maps show functional classification of roads.

(2) *Interstate Highways.* The interstate highway system was originally designed to be high-speed interregional connectors and it is a portion of the National Highway System (NHS). In urban and suburban areas, a large percentage of vehicular traffic is carried on the interstate highway system, rather than on the local arterials and streets.

(3) *State Routes.* The State highway system is described in the California Streets and Highway Code, Division 1, Chapter 2 and they are further defined in this manual in Topic 62.3, Highway Types which provides definitions for freeways, expressways, and highways.

81.5 Access Control

Index 62.3 defines a controlled access highway and a conventional highway. The level of access control plays a part in determining the design standards that are to be utilized when designing a highway. See Index 405.6 for additional access control guidance.

81.6 Design Standards and Highway Context

The design guidance and standards in this manual have been developed with the intent of ensuring that:

- Designers have the ability to design for all modes of travel (vehicular, bicycle, pedestrian, truck and transit); and,
- Designers have the flexibility to tailor a project to the unique circumstances that relate to it and its location, while meeting driver expectation.

Designers should balance the interregional transportation needs with the needs of the communities they pass through. The design of projects should, when possible, expand the options for biking, walking, and transit use. In planning and designing projects, the project development team should work with locals that have any livable policies as revitalizing urban centers, building local economies, and preserving historic sites and scenic country roads. The “Main Streets: Flexibility in Planning, Design and Operations” published by the Department should be consulted for additional guidance as should the FHWA publication “Flexibility in Highway Design”.

Early consultation and discussion with the Project Delivery Coordinator and the District Design Liaison during the project initiation document (PID) phase is also necessary to avoid issues that may arise later in the project development process. Design Information Bulletin 78 “Design Checklist for the Development of Geometric Plans” is a tool that can be used to identify and discuss design features that may deviate from standard.

Topic 82 - Application of Standards

82.1 Highway Design Manual Standards

(1) *General.* The highway design criteria and policies in this manual provide a guide for the engineer to exercise sound judgment in applying standards, consistent with the above Project Development philosophy, in the design of projects. This guidance allows for flexibility in applying design standards and approving design exceptions that take the context of the project location into consideration; which enables the designer to tailor the design, as appropriate, for the specific circumstances while maintaining safety.

The design standards used for any project should equal or exceed the minimum given in the Manual to the maximum extent feasible, taking into account costs (initial and life-cycle), traffic volumes, traffic and safety benefits, right of way, socio-economic and environmental impacts, maintenance, etc. Because design standards have evolved over many years, many existing highways do not conform fully to current standards. It is not intended that current manual standards be applied retroactively to all existing State highways; such is neither warranted nor economically feasible. However, when warranted, upgrading of existing roadway features such as guardrail, lighting, superelevation, roadbed width, etc., should be considered, either as independent projects or as part of larger projects. A record of the decision not to upgrade the existing non-standard mandatory or advisory features are to be provided through the exception process (See Index 82.2).

This manual does not address temporary construction features. It is recognized that the construction conditions encountered are so diverse and variable that it is not practical to set geometric criteria. Guidance for use of traffic control devices for temporary construction zones can be found in Part 6 – Temporary Traffic Control of the California Manual on Uniform Traffic Control Devices (California MUTCD). Guidance for the engineering of pavements in temporary construction zones is available in Index 612.6. In this manual, design standards and guidance are categorized in order of importance in development of a State highway system. See Index 82.4 for other mandatory procedural requirements.

(2) *Absolute Requirements.* Design guidance related to requirements of law, policy, or statute that do not allow exception are phrased by the use of “is required”, “without exception”, “are to be”, “is to be”, “in no event”, or a combination of these terms.

(3) *Controlling Criteria.* The FHWA has designated thirteen controlling criteria for selection of design standards of primary importance for highway safety, listed as follows: design speed, lane width, shoulder width, bridge width, horizontal alignment, vertical alignment, grade, stopping sight distance, cross slope, superelevation, horizontal clearance, vertical clearance and bridge structural capacity. All but the last of these criteria are also designated as geometric criteria.

The design standards related to the 12 geometric criteria are designated as mandatory standards in this manual (see Index 82.1(2) and Table 82.1A).

(4) *Mandatory Standards.* Mandatory design standards are those considered most essential to achievement of overall design objectives. Many pertain to requirements of law or regulations such as those embodied in the FHWA's 13 controlling criteria (see above). Mandatory standards use the word "shall" and are printed in **Boldface** type (see Table 82.1A).

(5) *Advisory Standards.* Advisory design standards are important also, but allow greater flexibility in application to accommodate design constraints or be compatible with local conditions on resurfacing or rehabilitation projects. Advisory standards use the word "should" and are indicated by <u>Underlining</u> (see Table 82.1B).

(6) *Decision Requiring Other Approvals.* There are design criteria decisions that are not bold or underlined text which require specific approvals from individuals to whom such decisions have been delegated. These individuals include, but are not limited to, District Directors, Traffic Liaisons, Project Delivery Coordinators or their combination as specified in this manual. These decisions should be documented as the individual approving desires.

(7) *Permissive Standards.* All standards other than absolute requirements, mandatory, advisory, or decisions requiring other approvals, whether indicated by the use of “should”, “may”, or “can” are permissive.

(8) *Other Caltrans Publications.* In addition to the design standards in this manual, see

Index 82.7 for general information on the Department's traffic engineering policy, standards, practices and study warrants.

Caution must be exercised when using other Caltrans publications which provide guidelines for the design of highway facilities, such as HOV lanes. These publications do not contain design standards; moreover, the designs suggested in these publications do not always meet Highway Design Manual Standards. Therefore, all other Caltrans publications must be used in conjunction with this manual.

(9) *Transportation Facilities Under the Jurisdiction of Others.* Generally, if the local road or street is a Federal-aid route it should conform to AASHTO standards; see Topic 308 – Cross Sections for Roads Under Other Jurisdictions. Occasionally though, projects on the State highway system involve work on adjacent transportation facilities that are under the jurisdiction of cities and counties. Some of these local jurisdictions may have published standards for facilities that they own and operate. The guidance in this manual may be applicable, but it was prepared for use on the State highway system. Thus, when project work impacts adjacent transportation facilities that are under the jurisdiction of cities and counties, local standards and AASHTO guidance must be used in conjunction with this manual to encourage designs that are sensitive to the local context and community values. Agreeing on which standards will be used needs to be decided early in the project delivery process and on a project by project basis.

82.2 Approvals for Nonstandard Design

(1) *Mandatory Standards.* Design features or elements which deviate from mandatory standards indicated herein require the approval of the Chief, Division of Design. This approval authority has been delegated to the District Directors for projects on conventional highways and expressways, and for certain other facilities in accordance with the current District Design Delegation Agreement. Approval authority for mandatory design standards on all other facilities has been delegated to the Project Delivery Coordinators except as noted in Table 82.1A where: (a) the mandatory standard has been delegated to the District Director and (b) the mandatory standards in Chapters 600 through 670 requires the approval of the State Pavement Engineer, or, (c) specifically delegated to the District Directors per the current District Design Delegation Agreements and may involve coordination with the Project Delivery Coordinator. See the HQ Division of Design website for the most current District Design Delegation Agreements.

The current procedures and documentation requirements pertaining to the approval process for those exceptions to mandatory design standards as well as the dispute resolution process are contained in Chapter 21 of the Project Development Procedures Manual (PDPM).

Design exception approval must be obtained pursuant to the instructions in PDPM Chapter 9.

The Moving Ahead for Progress in the 21st Century Act (MAP-21) of 2012 allowed significant delegation to the states by FHWA to approve and administer portions of the Federal-Aid Transportation Program. MAP-21 further allowed delegation to the State DOT's and in response to this a Stewardship and Oversight Agreement (SOA) document between FHWA and Caltrans was signed. The SOA outlines the process to determine specific project related delegation to Caltrans. In general, the SOA delegates approval of exceptions to mandatory design standards related to the 13 controlling criteria on all Interstate projects whether FHWA has oversight responsibilities or not to Caltrans. Exceptions to this delegation would be for projects of FHWA Division or Corporate interest which are determined on a project by project basis. See Index 43.2 for additional information. Consultation with FHWA should be sought as early in the project development process as possible. However, formal FHWA approval, if applicable, shall not be requested until the appropriate Caltrans representative has approved the design exception.

FHWA approval is not required for exceptions to "Caltrans-only" mandatory standards. Table 82.1A identifies these mandatory standards.

For local facilities crossing the State right of way see Index 308.1.

(2) *Advisory Standards*. The authority to approve exceptions to advisory standards has been delegated to the District Directors. A list of advisory standards is provided in Table 82.1B. Proposals for exceptions from advisory standards can be discussed with the District Design Liaison during development of the approval documentation. The responsibility for the establishment of procedures for review, documentation, and long term retention of approved exceptions from advisory standards has also been delegated to the District Directors.

(3) *Decisions Requiring Other Approvals*. The authority to approve specific decisions identified in the text are also listed in Table 82.1C. The form of documentation or other instructions are provided as directed by the approval authority.

(4) *Permissive Standards*. A record of deviation from permissive standards and the disclosure of the engineering decisions in support of the deviation should be documented and placed in the project file. This principle of documentation also applies when following other Division of Design guidance, e.g., Design Information Bulletins and Design Memos. The form of documentation and other instructions on long term retention of these engineering decisions are to be provided as directed by the District approval authority.

(5) *Local Agencies.* Cities and counties are responsible for the design decisions they make on transportation facilities they own and operate. The responsible local entity is delegated authority to exercise their engineering judgment when utilizing the applicable design guidance and standards, including those for bicycle facilities established by Caltrans pursuant to the Streets and Highways Code Sections 890.6 and 890.8 and published in this manual. For further information on this delegation and the delegation process, see the Caltrans Local Assistance Procedures Manual, Chapter 11.

82.3 FHWA and AASHTO Standards and Policies

The standards in this manual generally conform to the standards and policies set forth in the AASHTO publications, "A Policy on Geometric Design of Highways and Streets" (2011) and "A Policy on Design Standards-Interstate System" (2005). A third AASHTO publication, the latest edition of the "Roadside Design Guide", focuses on creating safer roadsides. These three documents, along with other AASHTO and FHWA publications cited in 23 CFR Ch 1, Part 625, Appendix A, contain most of the current AASHTO policies and standards, and are approved references to be used in conjunction with this manual.

AASHTO policies and standards, which are established as nationwide standards, do not always satisfy California conditions. When standards differ, the instructions in this manual govern, except when necessary for FHWA project approval (Index 108.3, Coordination with the FHWA).

The use of publications and manuals that are developed by organizations other than the FHWA and AASHTO can also provide additional guidance not covered in this manual. The use of such guidance coupled with sound engineering judgment is to be exercised in collaboration with the guidance in this manual.

82.4 Mandatory Procedural Requirements

Required procedures and policies for which Caltrans is responsible, relating to project clearances, permits, licenses, required tests, documentation, value engineering, etc., are indicated by use of the word "must". Procedures and actions to be performed by others (subject to notification by Caltrans), or statements of fact are indicated by the word "will".

82.5 Effective Date for Implementing Revisions to Design Standards

Revisions to design standards will be issued with a stated effective date. It is understood that all projects will be designed to current standards unless an exception has been approved in accordance with Index 82.2.

On projects where the project development process has started, the following conditions on the effective date of the new or revised standards will be applied:

- For all projects where the PS&E has not been finalized, the new or revised design standards shall be incorporated unless this would impose a significant delay in the project schedule or a significant increase in the project engineering or construction costs. The Project Delivery Coordinator or individual delegated authority must make the final determination on whether to apply the new or previous design standards on a project-by-project basis for roadway features.
- For all projects where the PS&E has been submitted to Headquarters Office Engineer for advertising or the project is under construction, the new or revised standards will be incorporated only if they are identified in the Change Transmittal as requiring special implementation.

For locally-sponsored projects, the Oversight Engineer must inform the funding sponsor within 15 working days of the effective date of any changes in mandatory or advisory design standards as defined in Index 82.2.

82.6 Design Information Bulletins and Other Caltrans Publications

In addition to the design standards in this manual, Design Information Bulletins (DIBs) establish policies and procedures for the various design specialties of the Department that are in the Division of Design. Some DIBs may eventually become part of this manual, while others are written with the intention to remain as design guidance in the DIB format. References to DIBs are made in this manual by the "base" DIB number only and considered to be the latest version available on the Department Design website. See the Department Design website for further information concerning DIB numbering protocol and postings.

Caution must be exercised when using other Caltrans publications, which provide guidelines for the design of highway facilities, such as HOV lanes. These publications do not contain design standards; moreover, the designs suggested in these publications do not always meet Highway Design Manual Standards. Therefore, all other Caltrans publications must be used in conjunction with this manual.

82.7 Traffic Engineering

The Division of Traffic Operations maintains engineering policy, standards, practices and study warrants to direct and guide decision-making on a broad range of design and traffic engineering features and systems, which are provided to meet the site-specific safety and mobility needs of all highway users.

The infrastructure within a highway or freeway corridor, segment, intersection or interchange is not "complete" for drivers, bicyclists and pedestrians unless it includes the appropriate traffic control devices; traffic safety systems; operational features or strategies; and traffic management elements and or systems. The presence or absence of these traffic elements and systems can have a profound effect on safety and operational performance. As such, they are commonly employed to remediate performance deficiencies and to optimize the overall performance of the "built" highway system.

For additional information visit the Division of Traffic Operations website at http://www.dot.ca.gov/hq/traffops/

Table 82.1A
Mandatory Standards

CHAPTER 100		**BASIC DESIGN POLICIES**
Topic 101		**Design Speed**
Index	101.1	Technical Reductions of Design Speed
	101.1	Selection of Design Speed - Local Facilities(2)
	101.1	Selection of Design Speed - Local Facilities - with Connections to State Facilities
	101.2	Design Speed Standards
Topic 104		**Control of Access**
Index	104.4	Protection of Access Rights(1)
CHAPTER 200		**GEOMETRIC DESIGN AND STRUCTURE STANDARDS**
Topic 201		**Sight Distance**
Index	201.1	Stopping Sight Distance Standards
Topic 202		**Superelevation**
Index	202.2	Standards for Superelevation
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Topic 203		**Horizontal Alignment**
Index	203.1	Horizontal Alignment - Local Facilities(2)
	203.1	Horizontal Alignment and Stopping Sight Distance
	203.2	Standards for Curvature – Minimum Radius
	203.2	Standards for Curvature – Lateral Clearance
Topic 204		**Grade**
Index	204.1	Standards for Grade - Local Facilities(2)
	204.3	Standards for Grade(2)
	204.8	Vertical Falsework Clearances(1)
Topic 205		**Road Connections and Driveways**
Index	205.1	Sight Distance Requirements for Access Openings on Expressways
Topic 208		**Bridges, Grade Separation Structures, and Structure Approach Embankment**
Index	208.1	Bridge Width
	208.4	Bridge Sidewalk (Width)
	208.10	Barriers on Structures with Sidewalks
	208.10	Bridge Approach Railings(1)
CHAPTER 300		**GEOMETRIC CROSS SECTION**
Topic 301		**Traveled Way Standards**
Index	301.1	Lane Width
	301.2	Class II Bikeway Lane Width(1)
	301.3	Cross Slopes – New Construction
	301.3	Cross Slopes – Resurfacing or widening
	301.3	Cross Slopes – Unpaved Roadway
	301.3	Algebraic Differences in Cross Slopes
Topic 302		**Shoulder Standards**
Index	302.1	Shoulder Width
	302.1	Shoulder Width with Rumble Strip
	302.2	Shoulder Cross Slopes -Bridge
	302.2	Shoulder Cross Slopes – Left
	302.2	Shoulder Cross Slopes – Paved Median
	302.2	Shoulder Cross Slopes - Right
Topic 305		**Median Standards**
Index	305.1	Median Width – Conventional Highways(1)
	302.2	Shoulder Cross Slopes - Right
Topic 305		**Median Standards**
Index	305.1	Median Width – Conventional Highways(1)

(1) Caltrans-only Mandatory Standard.

(2) Authority to approve deviations from this Mandatory Standard is delegated to the District Director.

(3) Authority to approve deviations from this Mandatory Standard is delegated to the State Pavement Engineer.

Table 82.1A
Mandatory Standards (Cont.)

	305.1	Median Width – Freeways and Expressways(1)
Topic 307		**Cross Sections for State Highways**
	Index 307.2	Shoulder Standards for Two-lane Cross Sections for New Construction
Topic 308		**Cross Sections for Roads Under Other Jurisdictions**
	Index 308.1	Cross Section Standards for City Streets and County Roads without Connection to State Facilities(2)
	308.1	Minimum Width of 2-lane Over-crossing Structures for City Streets and County Roads without Connection to State Facilities(1), (2)
	308.1	Cross Section Standards for City Streets and County Roads with Connection to State Facilities(1), (2)
	308.1	Two-Lane Local Road Lane Width for City Streets and County Roads within Interchange(2)
	308.1	Multi-Lane Local Road Lane Width for City Streets and County Roads within Interchange(2)
	308.1	Shoulder Width Standards for City Streets and County Roads Lateral Obstructions(2)
	308.1	Shoulder Width Standards for City Streets and County Roads with Curbs and Gutter(2)
	308.1	Minimum Width for 2-lane Overcrossing at Interchanges(2)
Topic 309		**Clearances**
	Index 309.1	Horizontal Clearances and Stopping Sight Distance
	309.1	Horizontal Clearances
	309.2	Vertical Clearances - Major Structures
	309.2	Vertical Clearances - Minor Structures
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(1) Caltrans-only Mandatory Standard.

(2) Authority to approve deviations from this Mandatory Standard is delegated to the District Director.

(3) Authority to approve deviations from this Mandatory Standard is delegated to the State Pavement Engineer.

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(1) Caltrans-only Mandatory Standard.
(2) Authority to approve deviations from this Mandatory Standard is delegated to the District Director.
(3) Authority to approve deviations from this Mandatory Standard is delegated to the State Pavement Engineer.

Table 82.1A
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(1) Caltrans-only Mandatory Standard.

(2) Authority to approve deviations from this Mandatory Standard is delegated to the District Director.

(3) Authority to approve deviations from this Mandatory Standard is delegated to the State Pavement Engineer.

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504.2	Freeway Ramp Profile Grades
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	110.10	Proprietary Items
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Topic 204		**Grade**
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Topic 205		**Road Connections and Driveways**
Index	205.1	Conversion of a Private Opening
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	208.10	Concrete Barrier Type 80
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	208.11	Deviations from Foundation and Embankment Recommendations
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	504.3	Enforcement Areas and Maintenance Pullouts – Required Enforcement Area
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* Authority to approve deviations from this "Decision Requirement" is delegated to the District Director.

Table 82.1C
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Topic 606		**Research and Special Designs**
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	606.1	Research and Experimentation – Special Designs
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CHAPTER 620		**RIGID PAVEMENT**
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	701.2	Locked Gates - Maintenance Force Use
	701.2	Locked Gates - Used by Utility Companies*
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Topic 853		**Pipe Liners and Linings for Culvert Rehabilitation**
	Index 853.4	Alternative Pipe Liner Materials
CHAPTER 870		**CHANNEL AND SHORE PROTECTION – EROSION CONTROL**
Topic 872		**Planning and Location Studies**
	Index 872.3	Site Consideration
Topic 873		**Design Concepts**
	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
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Table 902.3	Large Tree Setback Requirements on Conventional Highway Medians in Main Street Context
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Topic 1101	**General Requirements**
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* Authority to approve deviations from this "Decision Requirement" is delegated to the District Director.

CHAPTER 100
BASIC DESIGN POLICIES

Topic 101 - Design Speed

Index 101.1 - Highway Design Speed

(1) *General.* Highway design speed is defined as: "a speed selected to establish specific minimum geometric design elements for a particular section of highway". These design elements include vertical and horizontal alignment, and sight distance. Other features such as widths of pavement and shoulders, horizontal clearances, etc., are generally not directly related to highway design speed.

A highway carrying a higher volume of traffic may justify a higher design speed than a lower classification facility in similar topography, particularly where the savings in user operation and other costs are sufficient to offset the increased cost of right of way and construction. A lower design speed, however, should not be assumed for a secondary road where the topography is such that drivers are likely to travel at higher speeds.

It is preferable that the design speed for any section of highway be a constant value. However, during the detailed design phase of a project, situations may arise in which engineering, economic, environmental, or other considerations make it impractical to provide the minimum elements for other design standards (e.g., curve radius, stopping sight distance, etc.) established by the design speed. See Topic 82 for documenting localized exceptions to features preventing the standard design speed.

The cost to correct such restrictions may not be justified. Technically, this will result in a reduction in the effective design speed at the location in question. **Such technical reductions in design speed shall be discussed with and documented as required by the District approval authority or Project Delivery Coordinator depending upon the current District Design Delegation Agreement.**

Where a reason for limiting speed is obvious to approaching drivers or bicyclists, these users are more apt to accept a lower operating speed than where there is no apparent reason for it.

(2) *Selection.* Selecting the design speed for a highway is part of the Project Development Team process. See the Project Development Procedures Manual for additional guidance.

(a) Considerations--The chosen design speed, for a highway segment or project, needs to take into consideration the following:

- The selected design speed should be consistent with the operating speeds that are likely to be expected on a given highway facility. Drivers and bicyclists adjust their speed based on their perception of the physical limitations of the highway and its vehicular and bicycle traffic. In addition, bicycling and walking can be encouraged when bicyclists and pedestrians perceive an increase in safety due to lower vehicular speeds.

- In California the majority of State highway projects modify existing facilities. When modifying existing facilities, the design speed selected should reflect the observed motor vehicle speed (operating speed) or the anticipated operating speed upon completion of modifications. Generally the posted speed is a reliable indicator of operating speed although operating speeds frequently exceed posted speeds. Speed limits and speed zones are discussed in Chapter 2 of the California MUTCD, which include references to the California Vehicle Code.

  For existing limited access highways and conventional highways in rural areas other than Main Streets, the selected design speed for these higher-speed facilities typically is 15 to 20 mph higher than the observed motor vehicle speed (operating speed).

For existing lower-speed conventional highways in urban areas and rural highways that are Main Streets with observed or proposed operating speeds of 45 mph or less, the design speed should be selected to be consistent with the highway context which may discourage high-speed operating behavior. Select a design speed that is logical with respect to topography, operating speed (or anticipated operating speed if the corridor is being redesigned and the physical characteristics of the highway are being changed), adjacent land use, design volumes for all users, collision history, access control, and facility type.

- On projects where posted speeds or observational data is not available, the choice of design speed is influenced principally by whether the area is rural or urban, the character of terrain, economic considerations, environmental factors, type and anticipated volume of vehicular traffic, presence of non-motorized traffic, functional classification of the highway, existing and planned adjacent land use. A highway in level or rolling terrain justifies a higher design speed than one in mountainous terrain. As discussed under Topic 109, scenic values are also a consideration in the selection of a design speed.

(b) Freeways and Expressways--In addition to the considerations above, as high a design speed as feasible should be selected for use on freeways and expressways, which are higher-speed facilities.

(c) Conventional Highways

(1) *State Highways*. In addition to the considerations above, the existing and planned highway context in terms of area place type, land use, types of users, etc. influence the selection of the appropriate design speed and should be taken into account by the Project Development Team.

Consideration should also be given to Local Agency standards and transportation plans for the facility when selecting the design speed.

(2) *Local Streets or Roads.* **Local streets or roads within the State right of way, including facilities which will be relinquished after construction (such as frontage roads), shall have minimum design speeds conforming to AASHTO standards, as per the functional classification of the facility in question.** <u>If the local agency having jurisdiction over the facility in question maintains design standards that exceed AASHTO standards, then the local agency standards should apply.</u>

Where the local facility connects to a freeway or expressway (such as ramp terminal intersections), the design speed of the local facility shall be a minimum of 35 miles per hour. <u>However, the design speed should be 45 miles per hour when feasible.</u>

Every effort should be made to avoid decreasing the design speed of a local facility through the State's right of way, and all due consideration should be given to local plans to upgrade or improve the facility in the near future.

101.2 Highway Design Speed Standards

Table 101.2 shows appropriate ranges of design speeds that shall be used for the various types of facilities, place types, and conditions listed. For additional guidance, see Index 101.1(2).

Table 101.2
Vehicular Design Speed

Facility Type	Design Speed (mph)
LIMITED ACCESS HIGHWAYS	
Freeways and expressways in mountainous terrain	50-80
Freeways in urban areas	55-80
Freeways and expressways in rural areas	70-80
Expressways in urban areas	50-70
CONVENTIONAL HIGHWAYS (2)	
Rural	
Flat terrain	55-70
Rolling terrain	50-60
Mountainous terrain	40-50
Main Streets – Cities, Towns, and Community Centers	30-40
Urban	
Arterials - Throughways	40-60
Arterials - Main Streets and Regional/Community Centers	30-40
Downtowns and City Centers	30
LOCAL FACILITIES (Within State right of way)	
Facilities crossing a freeway or expressway, connecting to a conventional highway or traversing a State facility	AASHTO (1)
Facilities connecting to a freeway or expressway	$35^M/45^A$
	M=Mandatory A=Advisory

(1) If outside of State right of way and no specific local standards apply, the minimum design speed shall be 30 miles per hour.

(2) For conventional highways eligible or designated as State scenic highways, see Index 109.2

Topic 102 - Design Capacity & Level of Service

102.1 Design Capacity (Automobiles)

Design capacity (automobiles) is the maximum volume of vehicle traffic for which a projected highway can provide a selected level of service. Design capacity varies with a number of factors, including:

(a) Level of service selected.

(b) Width of lanes.

(c) Number of lanes.

(d) Presence or absence of shoulders.

(e) Grades.

(f) Horizontal alignment.

(g) Operating speed.

(h) Lateral clearance.

(i) Side friction generated by parking, drive ways, intersections, and interchanges.

(j) Volumes of trucks, transit, recreational vehicles, bicycles and pedestrians.

(k) Spacing and timing of traffic signals, and the required timing to accommodate pedestrian crossing

Level of Service (LOS) is largely related to speed and density among many variables. Freeways should be designed to accommodate the design year peak hour traffic volumes and to operate at a LOS determined by District Planning and/or Traffic Operations. For a rough approximation of the number of lanes required on a multilane freeway, use the following design year peak hour traffic volumes per lane at the specified LOS:

	Level of Service	Design Year Peak Hour Vehicle Traffic Volume (Average Automobiles Per Lane Per Hour)
Urban	C-E	1400-2400
Rural	C-D	1000-1850

For conventional highways and expressways, District Planning and Traffic Operations should be consulted.

Automobile traffic volumes can be adjusted for the effect of grades and the mix of automobiles, trucks, and recreational vehicles if a more refined calculation is desired. In those cases, consult the "Highway Capacity Manual", published by the Transportation Research Board.

102.2 Design Capacity and Quality of Service (Pedestrians and Bicycles)

Sidewalks are to accommodate pedestrians at a Level of Service (LOS) equal to that of vehicles using the roadway, or better. More detailed guidance on design capacity for sidewalks is available in the "Highway Capacity Manual" (HCM), published by the Transportation Research Board. The HCM also has guidance regarding LOS for bicycle facilities for both on- and off–street applications. The LOS for on-street bicycle facilities should be equal to that of vehicles using the roadway or better. The design of off-street bicycle facilities can use the LOS methodology in the HCM when conditions justify deviations from the standards in Chapter 1000.

Topic 103 - Design Designation

103.1 Relation to Design

The design designation is a simple, concise expression of the basic factors controlling the design of a given highway. Following is an example of this expression:

ADT (2015) = 9800 D = 60%

ADT (2035) = 20 000 T = 12%

DHV = 3000 V = 70 mph

ESAL = 4 500 000 TI_{20} = 11.0

CLIMATE REGION = Desert

The notation above is explained as follows:

ADT (2015) -- The average daily traffic, in number of vehicles, for the construction year.

ADT (2035) -- The average daily traffic for the future year used as a target in design.

CLIMATE REGION -- Climate Region as defined in Topic 615. In addition to establishing design requirements for the project, this information is used by the Resident Engineer during construction to determine which clauses in the Standard Specifications apply to the project.

DHV -- The two-way design hourly volume, vehicles.

D -- The percentage of the DHV in the direction of heavier flow.

ESAL -- The equivalent single axle loads forecasted for pavement engineering. See Topic 613.

T -- The truck traffic volume expressed as a percent of the DHV (excluding recreational vehicles).

TI_{20} -- Traffic Index used for pavement engineering. The number in the subscript is the pavement design life used for pavement design. See Index 613.3(3).

V -- Design speed in miles per hour.

Within a project, one design designation should be used except when:

(a) The design hourly traffic warrants a change in the number of lanes, or

(b) A change in conditions dictates a change in design speed.

(c) The design daily truck traffic warrants a change in the Traffic Index.

The design designation should be stated in Project Initiation Documents and Project Reports and should appear on the typical cross section for all new, reconstructed, or rehabilitation (including Capital Preventative Maintenance) highway construction projects.

103.2 Design Period

Geometric design of new facilities and reconstruction projects should normally be based on estimated traffic 20 years after completion of construction. With justification, design periods other than 20 years may be approved by the District Director with concurrence by the Project Delivery Coordinator.

For roundabout design period guidance, see Index 405.10.

Safety, Resurfacing, Restoration, and Rehabilitation (RRR), and operational improvement projects should be designed on the basis of current ADT.

Complimentary to the design period, various components of a project (e.g., drainage facilities, structures, pavement structure, etc.) have a design life that may differ from the design period. For pavement design life requirements, see Topic 612.

Topic 104 - Control of Access

104.1 General Policy

Control of access is achieved by acquiring rights of access to the highway from abutting property owners and by permitting ingress and egress only at locations determined by the State.

On freeways, direct access from private property to the highway is prohibited without exception. Abutting ownerships are served by frontage roads or streets connected to interchanges.

104.2 Access Openings

See Index 205.1 for the definition and criteria for location of access openings. The number of access openings on highways with access control should be held to a minimum. (Private property access openings on freeways are not allowed.) Parcels which have access to another public road or street as well as frontage on the expressway are not allowed access to the expressway. In some instances, parcels fronting only on the expressway may be given access to another public road or street by constructing suitable connections if such access can be provided at reasonable cost.

With the exception of extensive highway frontages, access openings to an expressway are limited to one opening per parcel. Wherever possible, one opening should serve two or more parcels. In the case of a large highway frontage under one ownership, the cost of limiting access to one opening may be prohibitive, or the property may be divided by a natural barrier such as a stream or ridge, making it necessary to provide an additional opening. In the latter case, it may be preferable to connect the physically separated portions with a low-cost structure or road rather than permit two openings.

104.3 Frontage Roads

(1) General Policy.

(a) Purpose--Frontage roads are provided on freeways and expressways to:

- Control access to the through lanes, thus increasing safety for traffic.
- Provide access to abutting land ownerships.
- Provide or restore continuity of the local street or road systems.
- Provide for bicycle and pedestrian traffic that might otherwise need to use the freeway.

(b) Economic Considerations--In general, a frontage road is justified on freeways and expressways if the costs of constructing the frontage road are less than the costs of providing access by other means. Right of way considerations often are a determining factor. Thus, a frontage road would be justified if the investment in construction and extra right of way is less than either the severance damages or the costs of acquiring the affected property in its entirety. Frontage roads may be required to connect parts of a severed property or to serve a landlocked parcel resulting from right of way acquisition.

(c) Access Openings--Direct access to the through lanes is allowable on expressways. When the number of access openings on one side of the expressway exceeds three in 1,600 feet, a frontage road should be provided (see Index 104.2).

(2) New Alignment. Frontage roads generally are not provided on freeways or expressways on new alignment since the abutting property owners never had legal right of access to the new facility. They may be provided, however, on the basis of considerations mentioned in (1) above.

(3) Existing Alignment. Where a freeway or expressway is developed parallel to an existing highway or local street, all or part of the existing roadway often is retained as a

frontage road. In such cases, if access to remainders of land on the side of the freeway or expressway right of way opposite the old road cannot be provided by other means, a frontage road must be constructed to serve the landlocked remainders or the remainders must be purchased outright. The decision whether to provide access or purchase should be based on considerations of cost, right of way impacts, street system continuity and similar factors (see (1) above).

(4) *Railroad Crossings.* Frontage roads on one or both sides of a freeway or expressway on new alignment, owing to safety and cost considerations, frequently are terminated at the railroad right of way. When terminating a frontage road at the railroad crossing, bicycle and pedestrian traffic still needs to have reasonable access through the community.

Any new railroad grade crossings and grade separations, and any relocations or alterations of existing crossings must be cleared with the railroad and approved by the PUC.

(5) *Frontage Roads Financed by Others.* Frontage roads which are not a State responsibility under this policy may be built by the State upon request of a local political subdivision, a private agency, or an individual. Such a project must be covered by an agreement under which the State is reimbursed for all construction, right of way, and engineering costs involved.

104.4 Protection of Access Rights

For proper control of acquired access rights, fencing or other approved barriers shall be installed on all controlled access highways except as provided in Index 701.2(3)(e).

104.5 Relation of Access Opening to a Median Opening

<u>Access openings should not be placed within 300 feet of a median opening unless the access opening is directly opposite the median opening.</u>

Details on access openings are given under Index 205.1.

104.6 Maintaining Local Community Access

When planning and designing a new freeway or expressway, the designer needs to consider the impacts of an access controlled facility on the local community. Closing non-expressway local road connections may negatively impact access for pedestrians, bicyclists and equestrians. A new facility may inadvertently sever local non-motorized access creating long out of direction travel. Designers need to coordinate with local agencies for access needs across an access controlled facility.

104.7 Cross References

(a) Access Control at Intersections at Grade (see Index 405.6).

(b) Access Control at Interchanges (see Index 504.8).

Topic 105 - Pedestrian Facilities

105.1 General Policy

The California Vehicle Code Section 21949 has stated a policy for the Department to provide safe and convenient travel for pedestrians. Conventional highways can be used by pedestrians. Although the Department will work to provide safe and convenient pedestrian travel on these highways, not all of these highways will contain sidewalks and walkways. Connections between different modes of travel should be considered when designing highway facilities, as all people may become pedestrians when transferring to a transit based facility. Pedestrian use near transit facilities should be considered during the planning phase of transportation improvement projects. See DIB 82 for accessibility guidance of pedestrian facilities. See also Topics 115 and 116 for guidance regarding designing for bicycle traffic.

105.2 Sidewalks and Walkways

The design of sidewalks and walkways varies depending on the setting, standards, and requirements of local agencies. Sidewalks are desirable on conventional highways and on other areas of State highway right of way to serve pedestrians when warranted by sufficient population, density and development.

Coordination with the local agency that the State highway passes through is needed to determine the appropriate time to provide sidewalks.

Most local agencies in California have adopted varying design standards for urban and rural areas, as well as more specific requirements that are applicable to residential settings, downtowns, special districts, and other place types. These standards are typically tied to zoning requirements for land use established by local agencies. These land use decisions should take into account the ultimate need for public right of way, including the transportation needs of bicyclists and pedestrians. <u>The minimum width of a sidewalk should be 8 feet between a curb and a building when in urban and rural main street place types. For all other locations the minimum width of sidewalk should be 6 feet when contiguous to a curb or 5 feet when separated by a planting strip</u>. Sidewalk width does not include curbs. See Index 208.4 for bridge sidewalks. Using the minimum width may not be enough to satisfy the actual need if additional width is necessary to maintain an acceptable Level of Service (LOS) for pedestrians. Note that street furniture, buildings, utility poles, light fixtures and platoon generators, such as window displays and bus stops, can reduce the effective width of sidewalks and likewise the LOS of the walkway. Also, adequate width for curb ramps and driveways are other important accessibility considerations.

See Index 205.3(6) and the Standard Plans for sidewalk requirements at driveways.

See Index 208.6 for information on pedestrian overcrossings and undercrossings and Index 208.4 for sidewalks on bridges.

"A Policy on Geometric Design of Highways and Streets", issued by AASHTO, and the "Highway Capacity Manual", published by the Transportation Research Board contain pedestrian LOS criteria. These are means of measuring the ability of the existing pedestrian facilities to provide pedestrian mobility and to determine the need for improvements or expansions. If adequate capacity is not provided, pedestrian mobility may be seriously impeded.

Traffic volume-pedestrian warrants for sidewalks or other types of walkways along highways have not been established. In general, whenever the roadside and land development conditions are such that pedestrians regularly move along a highway, those pedestrians should be furnished with a sidewalk or other walkway, as is suitable to the conditions. Sidewalks are typically within public right of way of the local agency or the State. When within the State highway right of way, the need for sidewalks becomes a shared interest, since the zoning, planned development, and growth are under the local agency's purview. The State may assume financial responsibility for the construction of sidewalks and walkways under the conditions described below. See the Project Development Procedures Manual for further discussion of the State's responsibility in providing pedestrian facilities.

(1) *Replacement in Kind.* Where existing sidewalks are to be disturbed by highway construction, the replacement applies only to the frontages involved and no other sidewalk construction is authorized except:

    (a) As part of a right of way agreement.

    (b) Where the safety or capacity of the highway will be improved.

(2) *Conventional Highways.* The roadway cross section usually provides areas for pedestrians. If the safety or capacity of the highway will be improved, the State may contribute towards the cost of building a pedestrian facility with a local agency project or fund it entirely with a State highway project. The city, county, or property owner whose adjacent development generated the pedestrian traffic may build sidewalks on State right of way under a permit in accordance with the route concept report.

(3) *Freeway and other Controlled Access Facilities.* Sidewalks should be built across the freeway right of way on overcrossings and through undercrossings where necessary to connect with existing or planned sidewalks. Construction of planned sidewalks should be imminent. Within the foregoing criteria, sidewalks can be part of the original project or added later when the surrounding area develops.

(4) *Overcrossing and Undercrossing Approaches.* Where sidewalks are planned on overcrossing

structures or under a structure, an area should be provided to accommodate future sidewalks.

(5) *School Pedestrian Walkways.* School pedestrian walkways may be identified along a route used by school pedestrians that is not limited to crossing locations, but includes where physical conditions require students to walk in or along rural or suburban roadways.

(6) *Frontage Roads.* Sidewalks may be built along frontage roads connecting local streets that would otherwise dead end at the freeways. Such sidewalks can be new or replacements of existing facilities. Sidewalks may not be needed on the freeway side of frontage roads except where connections must be made to pedestrian separations or other connections where appropriate.

(7) *Separated Cross Streets.* Sidewalks may be built on separated cross streets where reconstruction of the cross street is made necessary by the freeway project and where the criteria of paragraph (3) above apply.

(8) *Transit Stops.* Sidewalks should be built to connect transit stops to local streets.

(9) *Vehicular Tunnels*. Sidewalks and pedestrian facilities may be built as part of vehicular tunnels which do not require ventilation as part of the tunnel structure. Contact the Division of Engineering Services - Structure Design (DES-SD), regarding allowable conditions.

(10) *Maintenance.* The State is responsible for maintaining and replacing damaged sidewalks within the right of way except:

(a) Where the sidewalk was placed by a private party under encroachment permit that requires the permittee to maintain the sidewalk, but only if the original permittee still owns the abutting property.

(b) Where the city or county has placed nonstandard sidewalks with colored or textured surfaces, or meandering alignment. See Maintenance Manual for additional discussion on State's maintenance responsibilities regarding sidewalks.

105.3 Pedestrian Grade Separations

(1) Pedestrian grade separation takes the form of pedestrian overcrossings or undercrossings. These grade separations are suitable for crossing freeways, rivers, railroads, canyons and other obstacles for which no other crossing opportunities exist.

See Index 208.6 for design guidance for pedestrian and bicycle overcrossings and undercrossings.

The need for a pedestrian grade separation is based on a study of the present and future needs of a particular area or community. Each situation should be investigated and considered on its own merits. The study should cover pedestrian generating sources in the area, pedestrian crossing volumes, type of highway to be crossed, location of adjacent crossing facilities, circuity, zoning, land use, sociological and cultural factors, and the predominant age of persons expected to utilize the facility.

Pedestrian patterns should be maintained across freeway routes where these patterns have been previously established. Where vehicular crossings are inadequate for pedestrians, separate structures should be provided. In general, if a circuitous route is involved, a pedestrian separation may be justified even though the number of pedestrians is small.

State participation in the financing of pedestrian separations at ramp terminals is not normally justified because of the crash history at these locations. Exceptions to this general policy should be considered only in special circumstances where no less expensive alternative is feasible.

Where a pedestrian grade separation is justified, an overcrossing is preferred. Undercrossings tend to provide less visibility which provides more opportunities for vandalism and criminal activity. Consideration may be given to an undercrossing when specifically requested in writing by a local agency. Unobstructed visibility should be provided through the structure and approaches.

See Index 105.4 for discussion of provisions for persons with disabilities.

(2) *Financing.*

(a) Freeways--Where the pedestrian grade separation is justified prior to award of the freeway contract, the State should pay the full cost of the pedestrian facility. In some cases, construction of the separation may be deferred; however, where the need has been established to the satisfaction of the Department prior to award of the freeway contract, the State should pay the entire cost of the separation.

Local jurisdictions have control (by zoning and planning) of development that influences pedestrian traffic patterns. Therefore, where a pedestrian grade separation is justified after the award of a freeway contract, the State's share of the total construction cost of the separation should not exceed 50 percent. The State must enter into a cooperative agreement with the local jurisdiction on this basis.

(b) Conventional Highways--Grade separations are not normally provided for either cars or pedestrians on conventional highways. However, in those rare cases where pedestrian use is extensive, where it has been determined that placement and configuration of the grade separation will result in the majority of pedestrians using it, and where the local agency has requested in writing that a pedestrian separation be constructed, an overcrossing may be considered. The State's share of the total construction cost of the pedestrian facility should not exceed 50 percent. The State must enter into a cooperative agreement with the local jurisdiction on this basis.

105.4 Accessibility Requirements

(1) *Background.*

The requirement to provide equivalent access to facilities for all individuals, regardless of disability, is stated in several laws adopted at both the State and Federal level. Two of the most notable references are The Americans with Disabilities Act of 1990 (ADA) which was enacted by the Federal Government and took effect on January 26, 1992, and Section 4450 of the California Government Code.

(a) Americans with Disabilities Act Highlights.

- Title II of the ADA prohibits discrimination on the basis of disability by state and local governments (public entities). This means that a public entity may not deny the benefits of its programs, activities and services to individuals with disabilities because its facilities are inaccessible. A public entity's services, programs, or activities, when viewed in their entirety, must be readily accessible to and usable by individuals with disabilities. This standard, known as "program accessibility," applies to all existing facilities of a public entity.

- Public entities are not necessarily required to make each of their existing facilities accessible. Public entities may achieve program accessibility by a number of methods (e.g., providing transit as opposed to structurally accessible pedestrian facilities). However, in many situations, providing access to facilities through structural methods, such as alteration of existing facilities and acquisition or construction of additional facilities, may be the most efficient method of providing program accessibility.

- Where structural modifications are required to achieve program accessibility, a public entity with 50 or more employees is required to develop a transition plan setting forth the steps necessary to complete such modifications.

- In compliance with the ADA, Title 28 of the Code of Federal regulations (CFR) Part 35 identifies all public entities to be subject to the requirements for ADA regardless of funding source. It further states that the Uniform Federal Accessibility Standards (UFAS) and the Americans with Disabilities Act Accessibility Guidelines for Buildings and Facilities (ADAAG) are acceptable design guidelines that may be used. However, FHWA has directed Caltrans to use the ADAAG as the Federal design guidelines for pedestrian accessibility.

(b) California Government Code 4450 et seq. Highlights.

- Sections 4450 (through 4461) of the California Government Code require that buildings, structures, sidewalks, curbs, and related facilities that are constructed using any State funds, or the funds of cities, counties, or other political subdivisions be accessible to and usable by persons with disabilities.

(2) Policy.

It is Caltrans policy to:

- Comply with the ADA and the Government Code 4450 et seq. by making all State highway facilities accessible to people with disabilities to the maximum extent feasible. In general, if a project on State right of way is providing a pedestrian facility, then accessibility must be addressed.

(3) Procedures.

(a) The engineer will consider pedestrian accessibility needs in the Project Initiation Documents (PSRs, PSSRs, etc.) for all projects where applicable.

(b) All State highway projects administered by Caltrans or others with pedestrian facilities must be designed in accordance with the requirements in Design Information Bulletin 82, "Pedestrian Accessibility Guidelines for Highway Projects."

(c) The details of the pedestrian facilities and their relationship to the project as a whole should be discussed with the District Design Liaison for the application of DIB 82, the guidance of this manual, as well as other required design guidance.

ADA compliance must be recorded on the Ready-to-List certification for State-administered projects. Appropriate project records should document the fact that necessary review and approvals have been obtained as required above.

In addition to the above mentioned Design procedures, the District's have established procedures for certifying that the project "as-built" complies with the ADA standards in DIB 82 before a project can achieve Construction Contract Acceptance (CCA) or before the Notice of Completion is provided for a permit project.

105.5 Guidelines for the Location and Design of Curb Ramps

(1) Policy. On all State highway projects adequate and reasonable access for the safe and convenient movement of persons with disabilities are to be provided across curbs that are constructed or replaced at pedestrian crosswalks. This includes all marked and unmarked crosswalks, as defined in Section 275 of the Vehicle Code.

Access should also be provided at bridge sidewalk approaches and at curbs in the vicinity of pedestrian separation structures.

Where a need is identified at an existing curb on a conventional highway, a curb ramp may be constructed either by others under encroachment permit or by the State.

(2) Location Guidelines. When locating curb ramps, designers must consider the position of utilities such as power poles, fire hydrants, street lights, traffic signals, and drainage facilities.

<u>On new construction, two curb ramps should be installed at each corner as shown on the Standard Plans</u>. The usage of the one-ramp design should be restricted to those locations where the volume of pedestrians and vehicles making right turns is low. This will reduce the potential frequency of conflicts between

turning vehicles and persons with disabilities entering the common crosswalk area to cross either street.

Ramps and/or curb openings should be provided at midblock crosswalks and where pedestrians cross curbed channelization or median islands at intersections. Often, on traffic signalization, channelization, and similar projects, curbs are proposed to be modified only on portions of an existing intersection. In those cases, consideration should be given to installing retrofit curb ramps on all legs of the intersection.

(3) Ramp Design. Curb ramp designs should conform to current Standard Plans. See Index 105.4(3) for review procedures.

105.6 Pedestrian Crossings

There are various standards related to pedestrian crossings in this manual (e.g., the two curb ramps at each corner and pedestrian refuge island standards), as well as in DIB 82 (e.g., the curb ramp requirement) that depend on the existence of a pedestrian crossing as prescribed in the California Vehicle Code (CVC).

Pedestrian facilities that support pedestrian crossings occur at marked and unmarked crosswalks.

Per the CA MUTCD, a marked crosswalk is striped, including at midblock locations. An unmarked crosswalk is not striped and, per the CVC, depends on two elements: 1) it occurs at an intersection, and 2) it occurs where the sidewalk connects to the intersection. Without these two elements, there is no unmarked crosswalk.

Per the CVC, pedestrian crossings are provided across highways as marked or unmarked crosswalks, thereby requiring vehicles to yield to pedestrians (CVC 21950). Two examples in Figure 105.6 clarify the existence of unmarked crosswalks at "T" intersections, but may also apply to four legged intersections. This example is based on the following CVC citations:

- Section 275 - For the definition of crosswalk, see Index 62.4(4). Section 275 describes marked and unmarked crosswalks.
- Section 360 - A highway is a way or place of whatever nature, publicly maintained and open to the use of the public for purposes of vehicular travel. Highway includes street.
- Section 365 - An "intersection" is the area embraced within the prolongations of the lateral curb lines, or, if none, then the lateral boundary lines of the roadways, of two highways which join one another at approximately right angles or the area within which vehicles traveling upon different highways joining at any other angle may come in conflict.
- Section 530 - A "roadway" is that portion of a highway improved, designed, or ordinarily used for vehicular travel.
- Section 555 - A "sidewalk" is that portion of a highway, other than the roadway, set apart by curbs, barriers, markings or other delineation for pedestrian travel.

Topic 106 - Stage Construction and Utilization of Local Roads

106.1 Stage Construction

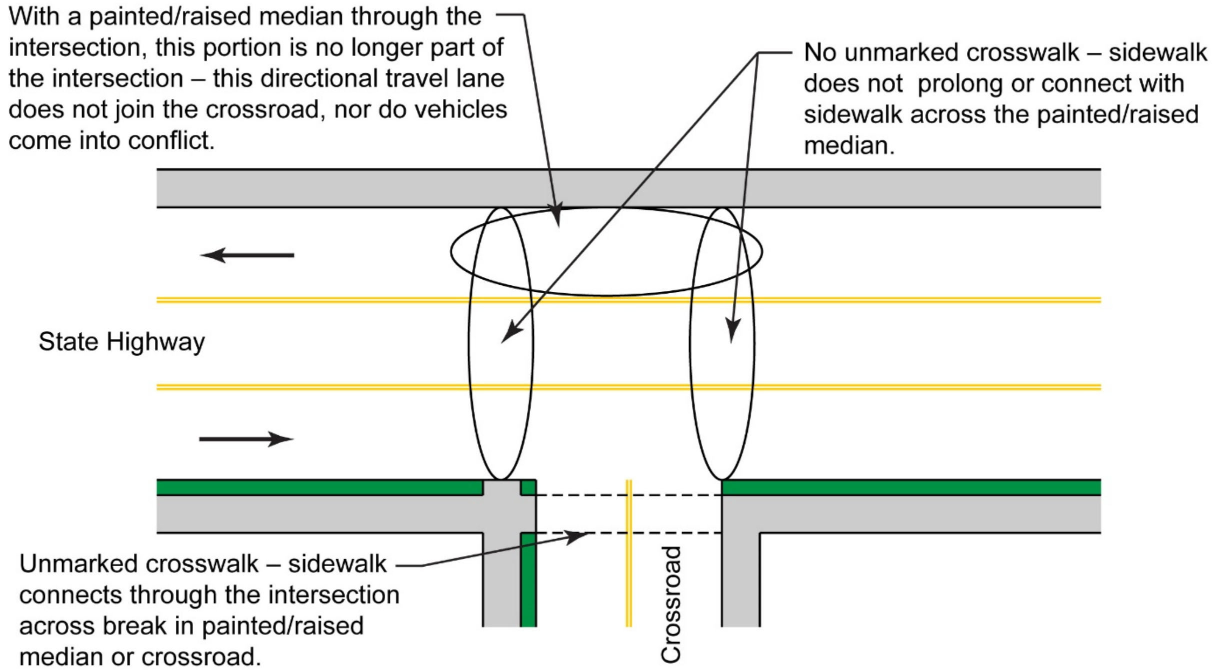
(1) Cost Control Measures. When funds are limited and costs increase, estimated project costs often exceed the amounts available in spite of the best efforts of the engineering staff. At such times the advantages of reducing initial project costs by some form of stage construction should be considered by the Project Delivery Team as an alternative to deferring the entire project. Stage construction may include one or more of the following:

(a) Shorten the proposed improvement, or divide it into segments for construction in successive years;

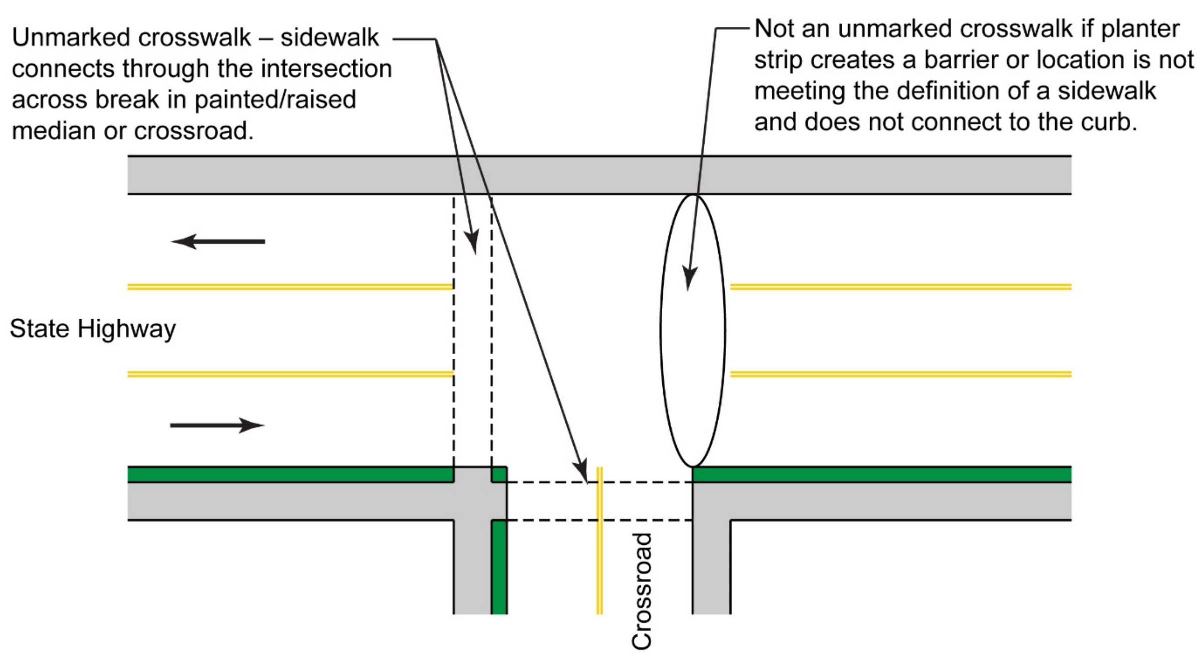
(b) Reduce number of lanes for initial construction. For example, a 4-lane freeway in a rural area with low current traffic volumes might be staged for two lanes initially with capacity adequate for at least 10 years after construction. Similarly, a freeway might be constructed initially four or six lanes wide with provision for future widening in the median to meet future traffic needs.

Figure 105.6

Typical Pedestrian Crossings at "T" Intersections



Example 1: State Highway with Partial Intersection



Example 2: State Highway Intersection

(c) Stage pavement structure. For flexible pavement, this could be done by reducing the surface course thickness with provision for a future overlay to bring the pavement to full design depth. For rigid pavement, the base and subbase layers could initially be built (if the base is built with HMA) and then overlaid later with a Portland cement concrete slab. In each case, life-cycle cost should be considered before using a staging option.

(d) Down scope geometric design features. This last expedient should be considered only as a last resort; geometric features such as alignment, grade, sight distance, weaving, or merging distance, are difficult and expensive to change once constructed. All nonstandard features need to comply with Index 82.2

A choice among cost reducing alternatives should be made only after weighing the benefits and disadvantages of each, particularly as they apply to interchange designs, which have a substantial effect on cost. See Index 502.3(2) for design considerations regarding freeway interchanges.

106.2 Utilization of Local Roads

In the construction of freeways or other highways by stages or construction units, it frequently becomes necessary to use portions of the local road system at one or more stages prior to completion of the whole route. Usually the local road is used as a traversable connection between the newly completed segment and the existing State highway.

Where such use of a local road is required, it may be handled by:

(a) Temporarily adopting the local road system as a traversable State highway, or

(b) Designating the local road system as a detour until the next or final stage is constructed.

(1) Temporary Adoption of Local Roads as State Routes. Temporary adoption of a local road system as a traversable route requires CTC action. Temporary adoption should be implemented where, for example, one unit of the freeway construction has been completed and the District wishes to route all users over the new roadway without waiting for completion of the next succeeding units, and the use of local roads is necessary to connect the freeway with the old State highway. Temporary adoption is useful where construction of the next freeway unit is a number of years in the future.

Such a temporary CTC adoption makes it legally possible to relinquish the old highway portion superseded by relocation.

Normally, the Department will finance any needed improvement required to accommodate all users during the period the local road system is a traversable State route. Financing by the local agency is not required. However, adoption of the local road by the CTC must precede State financing and construction of the proposed improvements.

When a local facility is adopted as a traversable route, the Department is responsible for all maintenance costs of the local facility unless otherwise provided for under the terms of a cooperative agreement. The Department normally would not assume maintenance until the road is in use as a connection or, when necessary, until the award of an improvement contract.

Formal concurrence of the local agency must be obtained before an adoption action is presented to the CTC.

If the local agency wants more improvements than are needed to accommodate all users during the period when the local road is used as a State highway connection, betterments are to be financed by the local agency. In such cases a cooperative agreement would be necessary to define the responsibilities of each party for construction and maintenance.

(2) Local Roads Used as Detours. In lieu of temporary adoption by the CTC, a local road may be designated a detour to serve as a connection between the end of State highway construction and the old State highway following completion of a State highway construction unit and pending completion of the next unit. Local road detours are useful if

the adjoining construction unit is scheduled in a few years or less and the local road connection is short and direct. Adoption by the CTC is not required when a local road is designated as a temporary detour.

Under Section 93 of the Streets and Highways Code, the Department can finance any needed improvements required to accommodate the detour of all users during the period the local road is utilized to provide continuity for State highway users. A cooperative agreement is usually required to establish terms of financing, construction, maintenance, and liability. If the local agency wants more than the minimum work needed to accommodate users on the local road during its use as a State highway, such betterments are to be financed by the local agency.

Section 93 also makes the Department responsible for restoration of the local road or street to its former condition at the conclusion of its use as a detour. The Department is responsible for all reasonable additional maintenance costs incurred by local agencies attributable to the detour. If a betterment is requested by the local agency as a part of restoration it should be done at no cost to the Department.

Topic 107 - Roadside Installations

107.1 Roadway Connections

<u>All connections to vista points, truck weighing or brake inspection stations, safety rest areas, park and ride lots, transit stations or any other connections used by the traveling public, should be constructed to standards commensurate with the standards established for the roadway to which they are connected</u>. On freeways this should include standard acceleration and deceleration lanes and all other design features required by normal ramp connections (Index 504.2). On conventional highways and expressways, the standard public road connection should be the minimum connection (Index 405.7).

<u>Only one means of exit and one means of entry to these installations should be allowed</u>.

107.2 Maintenance and Police Facilities on Freeways

Roadside maintenance yards and police facilities other than truck weighing installations and enforcement areas are not to be provided with direct access to freeways. They should be located on or near a cross road having an interchange which provides for all turning movements. This policy applies to all freeways including Interstate Highways.

Maintenance Vehicle Pullouts (MVPs) provide parking for maintenance workers and other field personnel beyond the edge of shoulder. This improves safety for field personnel by separating them from traffic. It also frees up the shoulder for its intended use. The need and location of MVPs should be determined by the PDT at project initiation. MVPs should only be provided if it has been determined that maintenance access from outside the state right of way through an access gate or a maintenance trail within the state right of way is not feasible. Where frequent activity of field personnel can be anticipated, such as at a signal control box (See Index 504.3 (2)(j)) or at an irrigation controller, the MVP should be placed upstream of the work site, so that maintenance vehicles can help shield field personnel on foot. If the controller or roadside feature is located within the clear recovery zone, relocating it outside the clear recovery zone should be considered (See Index 309.1). The shoulder adjacent to MVPs should be wide enough for a maintenance vehicle to use for acceleration before merging onto the traveled way. If adequate shoulder width is unattainable, sufficient sight distance from the MVP to upstream traffic should be provided to prevent maintenance vehicles from disrupting traffic flow. When considering drainage alongside a MVP, it is preferable to provide a flow line around the MVP rather than along the edge of shoulder to collect the drainage before the MVP. This will prevent ponding between the MVP and edge of shoulder. See Standard Plan H9 for a typical MVP layout plan and section detail.

107.3 Location of Border Inspection Stations

Other agencies require vehicles entering California to stop at buildings maintained by these agencies for inspection of vehicles and cargoes. No such building, parking area, or roadway adjacent to the parking area at these facilities should be closer than 30 feet from the nearest edge of the ultimate traveled way of the highway.

Topic 108 - Coordination With Other Agencies

108.1 Divided Nonfreeway Facilities

Per Section 144.5 of the Streets and Highways Code, advance notice is required when a conventional highway, which is not a declared freeway, is to be divided or separated into separate roadways, if such division or separation will result in preventing traffic on existing county roads or city streets from making a direct crossing of the State highway at the intersection. In this case, 30 days notice must be given to the City Council or Board of Supervisors having jurisdiction over said roads or streets.

The provisions of Section 144.5 of the Streets and Highways Code are considered as not applying to freeway construction, or to temporary barriers for the purpose of controlling traffic during a limited period of time, as when the highway is undergoing repairs, or is flooded. As to freeway construction, it is considered that the local agency receives ample notice, by virtue of the freeway agreement, of the manner in which all local roads will be affected by the freeway, and that the special notice would therefore be superfluous.

When the notice is required, a letter should be prepared and submitted to the appropriate authorities at least 60 days before road revision will occur. Prior to the submittal of the letter and before plans are completed, the appropriate authorities should be contacted and advised of contemplated plans. The timing of this notice should provide ample opportunity for consideration of any suggestions or objection made. In general, it is intended that the formal notice of intent which is required by law will confirm the final plans which have been developed after discussions with the affected authorities.

The PS&E package should document the date notice was given and the date of reply by the affected local agencies.

The Division of Design must be notified by letter as soon as possible in all cases where controversy develops over the closures to crossing traffic.

108.2 Transit Loading Facilities

(1) *Freeway Application.* These instructions are applicable to projects involving transit loading facilities on freeways as authorized in Section 148 of the Streets and Highways Code. Instructions pertaining to the provisions for mass public transportation facilities in freeway corridors, authorized in Section 150 of the Streets and Highways Code, are covered in other Departmental written directives.

(a) During the early phases of the design process, the District must send to the PUC, governing bodies of local jurisdictions, and common carriers or transit authorities operating in the vicinity, a map showing the proposed location and type of interchanges, with a request for their comments regarding transit loading facilities. The transmittal letter should state that transit loading facilities will be constructed only where they are in the public interest and where the cost is commensurate with the public benefits to be derived from their construction. It should also state that if the agency desires to have transit loading facilities included in the design of the freeway that their reply should include locations for transit stops and any supporting data, such as estimates of the number of transit passengers per day, which would help to justify their request.

(b) Public Meeting and Hearings. No public meeting or hearing is to be held when all of the contacted agencies respond that transit loading facilities are not required on the proposed freeway. The freeway should be designed without transit loading facilities in these cases.

Where any one of the agencies request transit loading facilities on the proposed freeway, the District should hold a public meeting and invite representatives of each agency.

Prior to the public meeting, the District should prepare geometric designs of the transit loading facilities for the purpose of making cost estimates and determining the feasibility of providing the facilities. Transit loading facilities must be approved by the District Director with concurrence from the Project Delivery Coordinator (see Topic 82 for approvals).

(c) Justification. General warrants for the provision of transit loading facilities in terms of cost or number of passengers have not been established. Each case should be considered individually because the number of passengers justifying a transit loading facility may vary greatly between remote rural locations and high volume urban freeways.

Transit stops adjacent to freeways introduce security and operational concerns that may necessitate relocating the stop at an off-freeway location. These concerns go beyond having a facility located next to high speed traffic, but also entail the pedestrian route to the facility through a low density area removed from the general public.

It may be preferable for patrons to board and leave the bus or transit facility at an off-freeway location rather than use stairways or ramps to freeway transit stops. Where existing highways with transit service are incorporated into the freeway right of way, it may be necessary to make provisions for bus service for those passengers who were served along the existing highway. This may be accomplished either by providing freeway bus and/or transit loading facilities or by the bus leaving and re-entering the freeway at interchanges. See "A Policy on Geometric Design of Highways and Streets", AASHTO, and “Guide for Geometric Design of Transit Facilities on Highways and Streets”, AASHTO for a discussion of transit design and bus stop guidelines.

(d) Reports. On projects where all the agencies contacted have expressed the view that transit stops are not needed, a report to the Division of Design is not required. However, a statement to the effect that the PUC, bus companies, and local governmental agencies have been contacted regarding transit stops and have made no request for their provisions should be included in the final environmental document or the PS&E submittal, whichever is appropriate.

For projects where one or more of the agencies involved have requested transit loading facilities either formally or informally during public meeting(s), a complete report should be incorporated in the final environmental document. It should include:

- A map showing the section of freeway involved and the locations at which transit loading facilities are being considered.
- A complete discussion of all public meetings held.
- Data on type of transit service provided, both at present and after completion of the freeway.
- Estimate of cost of each facility, including any additional cost such as right of way or lengthening of structures required to accommodate the facility.
- Number of transit trips or buses per day and the number of on and off passengers per day served by the transit stops and the number estimated to use the proposed facilities.
- District's recommendation as to the provision of transit loading facilities. If the recommendation is in favor of providing transit loading facilities, drawings showing location and tentative geometric designs should be included.

(e) The DES-Structure Design has primary responsibility for the structural design of transit loading facilities involving structures. See Index 210.7. See also DIB 82 for instructions on submitting rail and transit station plans to the Department of General Services – Division of the State Architect (DSA) for review and approval of pedestrian facilities with regard to accessibility features. Accessible paths of travel must be provided to all pedestrian facilities, including shelters, tables, benches, drinking fountains, telephones, vending machines, and information kiosks. The path of travel from designated accessible parking, if applicable, 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 use of the facility by individuals using wheelchairs, walkers or other mobility aids. See the Department of General Services, Division of the State Architect, as well as the California Department of Transportation enforce the California Building Code (Title 24) for the various on-site improvements.

(f) A cooperative agreement should be used to document the understanding between the Department and any local agency which desires a transit facility. The agreement covers items such as funding, ownership, maintenance, and legal responsibility.

(g) Detailed design requirements can be obtained from the transit authority having jurisdiction over the transit facility. See Index 504.3(6) for design standards related to bus loading facilities on freeways.

(2) Conventional Highway Application. This guidance is applicable to projects involving transit loading facilities on conventional highways as authorized in Section 148 of the Streets and highways Code. Instructions pertaining to the provisions for Bus Rapid Transit (BRT) in conventional highway corridors are covered in other Departmental policy and directives.

(a) The selection of transit facilities on conventional highways should follow the general outline as noted above for transit facilities on freeways. Transit facilities shall be approved by the District Director as part of the authorizing document (PSR/PR, PR, PSSR, etc.).

(b) A cooperative agreement should be used to document the understanding between the Department and any local agency which desires a transit facility. The agreement covers items such as funding, ownership, maintenance, and legal responsibility.

(c) Detailed design requirements can be obtained from the transit authority having jurisdiction over the transit facility.

(d) See also DIB 82 for instructions on submitting rail and transit station plans to the Department of General Services – Division of the State Architect (DS) for review and approval of pedestrian facilities with regard to accessibility features. Accessible paths of travel must be provided to all pedestrian facilities, including shelters, tables, benches, drinking fountains, telephones, vending machines, and information kiosks. The path of travel from designated accessible parking for persons with disabilities, if applicable, 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 use of the facility with wheelchairs, walkers and other mobility aides. See Topic 404 for guidance regarding the Design Vehicle, and Index 626.4(3) for structural section guidance for bus pads.

108.3 Commuter and Light Rail Facilities Within State Right of Way

(1) General. These facilities may cross or operate parallel to a highway or other multi modal facility owned and operated by the Department. The following guidance covers all rail facilities, and all transportation facilities owned

and operated by the Department. See the Project Development Procedures Manual for additional information and procedures regarding encroachments within State right of way. See Index 309.1(4) for high speed rail guidance.

(2) *Rail Crossings.* Ideally, rail crossings of transportation facilities should be grade separated. Grade separations must not impact the ability of the Department to operate and maintain its facilities, which includes the ability to expand the existing transportation facilities in the future. All rail crossings are to be approved by the District Director. See the California MUTCD for guidance regarding traffic controls for grade crossings.

(3) *Parallel Rail Facilities.* Rail facilities may be sited within Department right of way when feasible alternatives do not exist for separate facilities. As necessary, rail facilities may be located within the median. If rail facilities are located in the median, they must not impact the ability of the Department to reasonably operate and maintain its facilities, which includes the ability to expand the existing transportation facilities in the foreseeable future. All parallel rail facilities are to be approved by the District Director.

(4) *Design Standards*. Transit facilities are to be designed and constructed per the standards contained elsewhere in this manual and exceptions are to be documented as discussed in Chapter 80.

(5) *Cooperative Agreements*. The design and construction of rail facilities within the Department right of way should be covered in a cooperative agreement. Subsequent maintenance and operations requirements should be addressed in a maintenance agreement or encroachment permit as necessary.

108.4 Bus Loading Facilities

(1) *General.* A bus stop is a marked location for bus loading and unloading. Bus stops may be midblock, adjacent to, but before an intersection (near side) or adjacent to but after an intersection (far side). The far side location is preferred as pedestrians may cross the intersection behind the bus, allowing the bus to re-enter the travel stream following a break in traffic caused by the signal timing.

(2) *Design Standards*. Transit facilities are to be designed and constructed per the standards contained elsewhere in this manual and exceptions are to be documented as discussed in Chapter 80.

Bus stops and busbays (see Index 303.4(3) for busbays) should have pavement structures designed in accordance with Index 626.4(3). See the "Guide for Geometric Design of Transit Facilities on Highways and Streets", AASHTO, for guidance on the selection and design of transit loading facilities.

(3) *Cooperative Agreements.* Close coordination with the transit provider(s) is required for the successful design and operation of bus stops and other transit facilities.

108.5 Bus Rapid Transit

For the purpose of design and coordination, Bus Rapid Transit (BRT) is to be considered the same as commuter and light rail facilities with regards to approvals and design guidance.

BRT often makes use of the existing infrastructure for its operation within State right of way. As a joint user of the State right of way, BRT may not eliminate pedestrian or bicycle facilities. Because of potential conflicts, BRT facilities located on conventional highways and expressways should follow, as appropriate, the guidance for traffic control in the California MUTCD for light rail facilities. Transit Cooperative Report Program (TCRP) Report Numbers 90, 117 and 118 have additional guidance on BRT planning, design, and implementation. BRT located on freeways should be designed in accordance with the HOV Guidelines.

(1) *Design Standards*. Transit facilities are to be designed and constructed per the standards contained elsewhere in this manual, and exceptions are to be documented as discussed in Chapter 80.

(2) *Cooperative Agreements*. The design and construction of BRT facilities within the Department right of way should be covered in a cooperative agreement. Subsequent

maintenance and operations requirements should be addressed in a maintenance agreement or encroachment permit as necessary.

108.6 High-Occupancy Toll and Express Toll Lanes

(1) *General*. This guidance is applicable to projects involving High-Occupancy Toll (HOT) and Express Toll Lanes on freeways. These facilities are operated by a regional transportation agency or Caltrans under statutory authority or with the approval of the California Transportation Commission. The HOV Guidelines are to be consulted when considering the design and operation of these facilities.

(2) *Design Standards*. HOT and Express Toll Lane facilities are to comply with the standards contained elsewhere in this manual. Exceptions are to be documented as discussed in Chapter 80. Therefore, caution must be exercised when using other Department publications such as the HOV Guidelines if conflicts in design standards are identified.

(*3)* *Cooperative Agreements*. For HOT or Express lane facilities sponsored by a regional transportation agency, a cooperative agreement is to be used to document the understanding between the Department and the regional transportation agency. The agreement must address all matters related to design, construction, maintenance, and operation of the toll facility, including, but not limited to, liability, financing, repair, rehabilitation, and reconstruction. The regional transportation agency must also enter into an agreement with the California Highway Patrol that addresses all law enforcement matters related to the toll facility.

108.7 Coordination with the FHWA

FHWA representatives should be contacted as indicated by the Joint Stewardship and Oversight Agreement.

(1) *General.* As early in the design process as possible, FHWA should be kept informed of proposed activities on Federal-aid routes. See the Appendix of the Project Development Procedures Manual for a complete list of FHWA involvement.

(2) *Approvals.* The District Directors are responsible for obtaining formal FHWA approval for the following items on Federal-aid routes, see the Project Development Procedures Manual and the FHWA Joint Stewardship Oversight Agreement for a more complete list:

(a) Route Adoption. See the Project Development Procedures Manual for a discussion of procedures to be followed to NEPA and design approvals.

(b) Changes in access control lines, changes in locations of connection points, adding connection points, or deleting connection points on the Interstate System (even when no Federal money is involved).

(c) Addition of or changes in locked gates under certain conditions See Index 701.2.

(d) Partial interchanges on the Interstate system. See Index 502.2.

(e) Design-life on Interstates System projects.

Approximately twelve months prior to PS&E submittal, a project review should be arranged by the District with the Project Delivery Coordinator and, as required, the FHWA per the Stewardship & Oversight Agreement, see Index 43.2, to discuss nonparticipating items and unusual or special design features. The importance of early contact is emphasized to avoid delays when final plans are prepared.

For additional information, see the Project Development Procedures Manual.

Topic 109 - Scenic Values in Planning and Design

109.1 Basic Precepts

For any highway, having a pleasing appearance is an important consideration. Scenic values must be considered along with safety, utility, economy, and all the other factors considered in planning and design. This is particularly true of the many portions of the State Highway System situated in areas of natural beauty. The location of the highway, its alignment and profile, the cross section design, and other features should be in harmony with the setting.

109.2 Design Speed

The design speed should be carefully chosen as it is the key element which establishes standards for the horizontal alignment and profile of the highway. These requirements in turn directly influence how well the highway blends into the landscape. Scenic values, particularly in areas of natural scenic beauty must play a part along with the other factors set forth under Index 101.1 in selecting a design speed.

109.3 Aesthetic Factors

Throughout planning and design consider the following:

(a) The location of the highway should be such that the new construction will preserve the natural environment and will lead to and unfold scenic positions. In some cases, additional minor grading not required for roadbed alignment may expose an attractive view or hide an unsightly one.

(b) The general alignment and profile of the highway should fit the character of the area traversed so that unsightly scars of excavation and embankment will be held to a minimum. Curvilinear horizontal alignment should be coordinated with vertical curvature to achieve a pleasing appearance.

(c) Existing vegetation (e.g., trees, specimen plants, diminishing native species or historical plantings) should be preserved and protected to the maximum extent feasible during the planning, design, and construction of transportation projects. Whenever specimen or mature trees are present, especially in forested areas, a tree survey should be made to provide accurate data on the variety, condition, location, size, and ground elevations of trees affected.

(d) Appropriate replacement planting should be provided when existing planting is removed. When native or specimen trees are removed, replacement planting should reflect the visual importance of the plantings lost. Where the visual impact of tree removal is substantial, replacement with large transplants or specimen size trees may be appropriate. If not, an appropriate quantity of smaller replacements may be required to ensure eventual survival of an adequate number of plants.

    Provisions for watering and establishment of replacement planting should also be considered. The District Landscape Architect should be consulted early in the planning and design process so that appropriate conservation and revegetation measures are incorporated.

(e) Existing vegetation such as trees or large brush may be selectively thinned or removed to open up scenic vistas or provide a natural looking boundary between forest and cleared areas. Vegetation removal for aesthetic purposes should be undertaken only with the concurrence of the District Landscape Architect.

(f) Vista points should be provided when views and scenery of outstanding merit occur and feasible sites can be found. (See Topic 904 for site selection criteria.)

(g) Whenever feasible, wide medians and independent roadways should be provided on multilane facilities as these features add scenic interest and relieve the monotony of parallel roadways.

(h) Bridges, tunnels, and walls merit consideration in lieu of prominent excavation and embankment slopes when costs of such alternates are not excessive.

(i) Slopes should be flattened and rounded whenever practical and vegetation provided so that lines of construction are softened.

(j) Structures should be located and designed to give the most pleasing appearance.

(k) Scars from material sites should be avoided. Planting compatible with the surroundings should be undertaken to revegetate such scars when they are unavoidable.

(l) Drainage appurtenances should be so located that erosion, sumps, and debris collection areas are hidden from view or eliminated when site conditions permit.

(m) Interchange areas should be graded as flat as reasonable with slope rounding and contouring to provide graceful, natural looking appearance. The appearance can be further enhanced by planting a vegetative cover appropriate to the locality, being careful to maintain driver visibility.

(n) In locations where graffiti has been excessive, concepts such as limiting accessibility, planting, and surface treatments should be considered to deter graffiti.

(o) Roadsides should be designed to deter weed growth along the traveled way, and to provide for mechanical litter collection.

Topic 110 - Special Considerations

110.1 Design for Overloaded Material Hauling Equipment

Sometimes bid costs can be reduced by allowing the hauling of overloads on a construction contract. The savings may warrant designing structures and structural sections of new roadways to carry the heavier loads and also reconstructing roadbeds used by overloaded material hauling equipment.

In general, hauling of overloads is restricted to the project limits. However, overloads are permitted on portions of existing highways which are to be abandoned, repaired or reconstructed with a new structural section, if the overloads do not affect the design of the reconstructed structural section.

Any overload requirements should be determined before detailed plans are prepared. The District should request from the Division of Engineering Services – Structures Design (DES - SD) the estimated additional cost of the structures to carry overloads and use this information in making economic comparisons.

Factors to be considered in making the comparisons should include the costs of strengthening structures, haul costs, amount of material to be hauled, repair or reconstruction of structural sections, construction of separate haul roads or structures, strengthening of the new structural section, sequence of construction operations, and other pertinent factors. In some cases, consideration should be given for requiring the contractor to construct a separate haul structure over a heavily traveled surface street when large quantities of material are involved.

The comparison and all factors leading to the decision should be complete, fully documented, and retained in the project files.

The design of structures for overloads will normally be governed by one of the following categories:

(1) *Category 1.* Structures definitely planned to carry overloads. This category should be used only when the structures are to be constructed under a separate contract prior to a grading contract and the estimated savings in grading costs exceed the extra structure costs. The District must request the DES - SD to design for the permissible overloading.

(2) *Category 2.* Structures which are designed to allow the contractor the option of strengthening to carry overloads. The contract plans will include alternative details for strengthening the structure and the contractor can decide at the time of bidding whether to haul around the structure, build his own haul road structures, use "legal load" equipment on the unstrengthened structure, or construct the structure in accordance with the strengthened alternative design. The District should notify the DOS regarding structures to have optional designs. Undercrossings, overheads, separations, and stream crossings are most likely to be in this category.

(3) *Category 3.* Structures which will not be designed to carry overloads. Most overcrossing, ramp, and frontage road structures are in this category.

The District should consult with the DOS early in the design phase when determining the design overload category of each bridge in the project. Each case where hauling of overloads is permitted must be specifically described in the Special Provisions. Each structure designed under Categories 1 and 2 must also be designated in the Special Provisions. The design load must not exceed the weight limitation of Section 7-1.02, "Weight Limitations", of the Standard Specifications. The District Director or designated representative must approve the overload category for each structure.

110.2 Control of Water Pollution

Water pollution related to the construction of highways and to the drainage of completed highways should be limited to the maximum extent practicable. This objective should be considered from the early planning, through the detailed design phase, to the end of construction of each project.

Proposed alterations of existing drainage patterns and creation of disturbed soil areas should consider the potential for erosion and siltation. Where interdisciplinary analysis (engineering, biology, geology, chemical) indicates that harmful physical, chemical, or biological pollution of streams, rivers, lakes, reservoirs, coastal waters, or groundwater may occur, preventive measures and practices will be required. These measures include temporary erosion control features during construction, scheduling of work, as well as the permanent facilities to be built under the contract. The control of erosion associated with permanent drainage channels and ditches is covered in Chapter 860, Open Channels.

The Department's Project Planning and Design Guide identifies the procedures and practices to be employed in order for projects to comply with the Storm Water Management Plan and the National Pollutant Discharge Elimination System Permit, issued by the State Water Resources Control Board.

Districts must initiate contact with the appropriate agencies responsible for water quality as early as feasible in development of transportation projects to ensure full identification of pollution problems, and to ensure full cooperation, understanding, and agreement between the Department and the other agencies. The agencies to be contacted will vary from project to project depending on the nature of the project, the aquatic resources present, and the uses of the water. The agencies that may be interested in a project include but are not limited to the following: U.S. Army Corps of Engineers, U.S. Fish and Wildlife Service, U.S. Environmental Protection Agency, California Regional Water Quality Control Boards, California Department of Fish and Game, Flood Control Districts, and local water districts. The District Environmental Unit can provide assistance in determining which agencies should be contacted.

Recommendations for mitigation measures or construction and operational controls contained in the project's Storm Water Data Report should receive full consideration in the development of the project. The Department is legally bound to comply with the appropriate permits as outlined in the California Permit Handbook. The Department is also legally bound to comply with any water quality mitigation measures specified in the project's environmental document. Plans and specifications should reflect water quality protection measures in a manner that is enforceable in contracts.

On almost all projects, early contact should be established between the District project development personnel, Landscape Architecture, biologists, geologists, and other specialists available in the Headquarters Environmental Program, the Division of Engineering Services (DES) Office of Structural Foundations, FHWA, or other Districts, to ensure optimum development of water quality control measures.

Because siltation resulting from erosion is recognized as a major factor in water pollution, continuous efforts should be made to improve erosion control practices.

(1) Project Planning Phase. When project planning studies are started, consideration should be given to the items in the following list:

(a) Identify all waters in the vicinity of a highway project which might affect construction, maintenance and operational activities.

The environmental factors that might affect preconstruction activities should be looked into for the benefit of the resident engineer and contractor. An example would be relocation of drilling of pile foundations in a sensitive stream to prevent possible impacts.

(b) Identify for each project all waters, both fresh and saline, surface and underground, where water quality may be affected by the proposed construction.

(c) Determine if any watersheds, aquifers, wells, reservoirs, lakes, or streams are sources for domestic water supplies.

(d) Determine if any sensitive fishery, wildlife, recreational, agricultural, or industrial aquatic resources are located in the vicinity of the project.

(e) Consider possible relocation or realignment that could be made to avoid or minimize the possibility of pollution of existing waters.

(f) Identify variations in the erosive characteristics of the soils in the area, and consider relocation or grade changes that would minimize erosion.

(g) Where possible, avoid unstable areas where construction may cause future landslides.

(h) Identify construction season preference of regulatory agencies.

(i) Evaluate the need for additional right of way to allow for flatter, less erosive slopes.

(2) *Design Phase.* During the design phase, the items listed above should again be considered. More specific items for consideration are presented in the following checklist:

(a) Provide for the preservation of roadside or median vegetation beyond the limits of construction by special provisions and depiction on the plans.

(b) Design slopes as flat as is reasonable with slope rounding, landforming/geomorphic grading, contouring, or stepping to minimize erosion and to promote plant growth. Consider retaining walls when practical to reduce slope length and steepness. Include standard special provisions or approved special provisions which will require the contractor to remove or excavate, stockpile, and apply topsoil and/or duff on the final slope to promote plant growth. For information on landforming/geomorphic grading see: http://www.dot.ca.gov/hq/LandArch/webinars/index.htm and work with district landscape architecture.

(c) Provide erosion control to all soil areas to be disturbed by construction activities. Consider the need to require the contractor to apply permanent erosion control in phases, as slopes become substantially complete, instead of allowing all erosion control to be applied at the end of the construction project. Prior to winterizing the project, the designer must plan for temporary erosion control on slopes not substantially complete. Native plants should be considered for all plantings.

If a highway planting project is anticipated immediately following roadway construction, disturbed soil areas cannot be left unprotected. The use of mulch could be considered as an erosion control method during the interim. Contact the District Landscape Architect for assistance.

(d) When planning for temporary erosion control, consider the use of vegetation, mulches, fiber mats, fiber rolls, netting, dust palliatives, crust forming chemicals, silt fences, plastic sheets or any other procedure that may be necessary to prevent erosion. The District Storm Water Coordinator, District Landscape Architect, and the District Storm Water Unit can assist in the selection and design of temporary erosion control measures.

(e) Design overside drains, surface, subsurface, and cross drains so that they will discharge in locations and in such a manner that surface and subsurface water quality will not be affected. The outlets may require aprons, bank protection, desilting basins, or energy dissipators.

(f) Provide for adequate fish passage through highway culverts or under bridges when necessary to protect or enhance fishery resources.

(g) Provide bank protection where the highway is adjacent to rivers, streams, lakes, or other bodies of water.

(h) Where required, provide slope protection or channel lining, energy dissipators, etc. for channel changes.

(i) Where the State has made arrangements for materials, borrow, or disposal sites, grading plans should be provided and revegetation required. Special provisions should require the contractor to furnish plans for grading and replanting of sites.

(j) Check right of way widths for adequate space to reduce slope gradients and minimize slope angles, for rounding at tops of cuts and bottoms of fills, for adequate slope protection ditches and for incorporation of treatment control measures (e.g., infiltration basins, detention basins, traction sand traps). Also consider right of way or encroachment rights for temporary work such as desilting basins, stream diversion, or stream crossing protection.

(k) All ditches should be designed to minimize erosion. These treatments include but are not limited to grass lining, fiber mats, rock lining (with or without geotextile underlayment), and paving. The District Hydraulics Unit can assist with the selection and design of ditch treatment. Consideration should be given to using soil stabilization materials in median ditches or other wide drainage areas that cannot be vegetated.

(l) Temporary construction features for water pollution control that can be predicted should be made a part of the plans, specifications, and contract pay items. Such items as mulching and seeding of slopes, berms, dikes, ditches, pipes, dams, silt fences, settling basins, stream diversion channels, slope drains, and crossings over live streams should be considered. Since all contingencies probably cannot be foreseen, supplemental work funds should be set up for each project. Pay items for temporary erosion control should not be adjusted for increased or decreased quantity.

(m) Special consideration should be given to using vegetated ditches to remove highway runoff pollutants. The District Hydraulics and Landscape Architecture Units can provide assistance in designing and constructing vegetated ditches.

(n) Mandatory order of work clauses sometimes result in increased costs or longer time limits, but they must be considered where their use would eliminate the expense of temporary construction or where they result in earlier protection of erodible areas, or improved handling of site runoff.

(3) Abandonment and Destruction of Water Wells. The abandonment and destruction of water wells within the highway right of way must be handled in accordance with requirements established by statute and by agreement with the Department of Water Resources (DWR) to avoid pollution of underground water and ensure public safety. Sections 13700 to 13806 of the California Water Code deal, in general, with the construction and destruction of wells. Section 24400 to 24404 of the Health and Safety Code require that abandoned wells be covered, filled, or fenced for safety reasons. Statewide standards for construction, maintenance and destruction of water wells, monitoring wells and cathodic protection wells have been issued by the California DWR in Bulletin 74 - 81, "Water Well Standards: State of California", dated December, 1981, and Bulletin 74 - 81", dated January, 1990. Pursuant to these standards and interagency agreement with DWR, the following procedures are to be followed to determine requirements for abandonment and destruction of wells within State highway rights of way.

(a) Before producing water wells within the highway right of way are abandoned, a determination should be made of the possible future uses of the wells. Such future uses include landscape irrigation, roadside rests, vista points, maintenance facilities, truck weighing facilities, and others. Also see Index 706.4.

(b) The District Project Development and Right of Way Branches determine the location of water wells that will be affected by highway construction on a project basis.

(c) The District submits a letter to the Director, Department of Water Resources, 1416 Ninth Street, Sacramento, CA. 95814 Attention: Water Resources Evaluation Section, Division of Resources Development, listing the wells to be abandoned and any information that may be known about them. The letter should include the scheduled PS&E date and the anticipated advertising date for the project. Two copies of a map, or maps, showing the location of each well accurately enough so it can be located in the field should be included with the letter. A copy of this package should also be provided to Headquarters Construction.

(d) DWR will investigate the wells and write a report recommending procedures to be used in destruction of the wells within the highway right of way. The interagency agreement provides for reimbursement of the DWR's cost for these investigations and reports.

(e) DWR will forward its report to the District.

(f) Provisions for destruction of abandoned wells occasioned by highway construction and planting projects must be included in the District PS&E report. The work, usually done by filling and sealing, normally should be included in the contract Special Provisions. Steps must be taken to insure that wells are left in a safe condition between the time the site is acquired by the State and the time the well is sealed.

(g) In some cases, local ordinances or conditions will require the filling and sealing of the well prior to the highway contract in order to leave the well in a safe condition.

(h) The contractor who does the work to abandon the well must file the Notice of Intent (Form DWR 2125) and the Water Well Drillers Report (Form DWR 188) required by the Department of Water Resources.

(i) Also, under California Water Code Section 13801, after January 15, 1990, all cities and counties are required to have adopted ordinances that require prior acquisition of permits for all well construction, reconstruction and destruction and requiring possession of an active C-57 contractors license as the minimum qualification for persons permitted to work on wells.

(4) Summary. To prevent pollution of all waters that could be affected by a highway construction project, it is desirable to avoid involvement with the water or avoid the construction of erodible features. Since it is seldom possible to avoid all such features, the design of effective erosion and sediment control measures should be included with the project. Material resulting from erosion should either be discharged in locations where no negative environmental impacts will occur, or be deposited in locations that are accessible to maintenance forces for removal. District Landscape Architecture can provide technical assistance in assessing the impacts of erosion and in designing erosion control features.

Project Development personnel should ensure that all aspects of erosion control and other water quality control features considered during design are fully explained to the Resident Engineer. Such data is essential for review of the contractor's water pollution control program. Judgment must be used in differentiating between planned temporary protection features and work which the contractor must perform in order to fulfill their responsibility to protect the work from damage.

To reduce contract change orders and ensure erosion control goals are met, important protection should not be left to the contractor's judgment. It is desirable that all predictable temporary protection measures be incorporated in the plans and specifications and items for payment included in the contract items of work.

Topsoil should be stripped, stockpiled, and restored to disturbed slopes because existing soil nutrients and native seeds contained within the topsoil are beneficial for establishing vegetative cover and controlling erosion.

In addition, the abandonment of water wells must be given special attention in accordance with Section (3) above.

110.3 Control of Air Pollution

Air pollution associated with the construction of highways and to completed highway facilities should be held to the practical minimum. The designer should consider the impacts of haul roads, disposal sites, borrow sites, and other material sources in addition to construction within the highway right of way.

(1) *Control of Dust.* Many of the items listed under Index 110.2, Control of Water Pollution, are applicable to dust control. Consideration should be given to these items and additional material presented in the following list:

(a) See Index 110.2(2)(a), (c), (d), (k) and (n).

(b) Flat areas not normally susceptible to erosion by water may require erosion control methods such as planting, stabilizing emulsion, protective blankets, etc., to prevent wind erosion.

(c) Cut and or fill slopes can be sources of substantial wind erosion. They will require planting or other control measures even if water erosion is only a minor consideration.

(d) In areas subject to dust or sand storms, vegetative wind breaks should be considered to control dust. Use of soil sealant may also be considered.

(e) Special provisions should be used requiring the contractor to restore material, borrow, or disposal sites, and temporary haul roads to a condition such that their potential as sources of blowing dust or other pollution is no greater than in their original condition. Work for this purpose that can be predicted should be made a part of the PS&E, which should require submission of the contractors plan for grading, seeding, mulching or other appropriate action.

(f) Stockpiling and respreading topsoil may speed revegetation of the roadside and reduce wind erosion.

(2) *Control of Burning.* Health and Safety Code provisions and rules issued by Air Pollution Control Boards will preclude burning on most highway projects. Off-site disposal of debris must not create contamination problems and should not be specified simply as an expedient resolution of the problem without imposing adequate controls on how such disposal site is to be handled. Designers should seek disposal site locations within the right of way where it will be permissible to dispose of debris. Proper procedures, including compaction and burial, should be specified. Debris should not be disposed of within the normal roadway. Burying within the right of way should be done in such a fashion that the layers of debris will not act as a permeable layer or otherwise be detrimental to the roadway. Acceptable alternates based on economic, aesthetic, safety, and other pertinent considerations should be included in the contract if possible.

On projects where burning will not be permitted and disposal of debris within the right of way is not possible, optional disposal sites should be made available. Information on such site arrangements should be made available in the "Materials Information" furnished to prospective bidders. Reference is made to the applicable portion of Index 111.3 and 111.4 for handling this requirement. Special requirements for disposal of debris and final appearance of the disposal site should be covered in the Special Provisions. The intent of this instruction is that the designer should make sure that prospective bidders have adequate information on which to make a realistic bid on clearing and grubbing.

When feasible, tree trunks, branches, and brush should be reduced to chips and incorporated with the soil, spread on fill slopes, used as a cover mulch or disposed of in other ways compatible with the location. In forest areas where they will not look out of place, limbs and trunks of trees that are too large for chipping may be limbed and cut to straight lengths and the pieces lined up at the toes of the slope. An earth cover may be necessary for aesthetic reasons, or to reduce fire hazards. Under certain conditions salvage of merchantable timber may be desirable, or may be required by right of way commitments. Whenever merchantable timber is to be salvaged, appropriate specifications should be provided. Stumps and unsightly clumps of debris should be chipped or buried in areas where they will not create future problems.

Care should be taken not to block drainage or to interfere with maintenance operations.

Before proposing chipping as the method of disposal, the designer should investigate to determine if plant disease or insect pests will be spread to disease-free or insect-free areas. Procedures to decontaminate such chips before use should be included in the contract if necessary. Designers should seek advice from local experts and County Agricultural Extension Offices to determine the extent of such problems and the procedures and chemicals to be specified.

The U.S. Forest Service and the State Division of Forestry should be contacted during the design stage to ascertain the requirements that these agencies will make upon any disposal methods to be used in areas under their control.

It will be noted that under certain limited conditions the prohibition against burning may be eliminated from the Special Provisions.

There will be some areas of the State where Air Pollution Control Boards may consider issuing a permit for open burning where the effect on air quality is expected to be negligible and few if any residents would be affected. The individual situation should be studied and appropriate special provisions prepared for each project to fully cover all possible methods of disposal of debris that will be available to the contractor.

The local Air Pollution Control Board should be contacted to determine the current regulations.

(3) *Summary*. Special consideration should be given to the direction of prevailing winds or high-velocity winds in relation to possible sources of dust and downwind residential, business, or recreational areas. Every practical means should be incorporated in the design of the highway and in the provisions of the contract to prevent air pollution resulting from highway construction and operation.

110.4 Wetlands Protection

The Nation's wetlands are recognized on both the Federal and State level as a valuable resource. As such, there have been several legislative and administrative actions which provide for special consideration for the preservation of wetlands. These are embodied on the Federal level in Executive Order 11990, DOT Order 5660.1A, Section 404 of the Clean Water Act, including Section 404(b)(1) guidelines, and the NEPA 404 Integration Process for Surface Transportation Projects, and the August 24, 1993 Federal Wetlands Policy. Wetlands are covered on the State level by the Porter-Cologne Water Quality Act and the Resources Agency's Wetlands Policy. The District Environmental Unit can provide assistance with permitting strategies, identifying wetlands, determining project impacts, and recommending mitigation measures, in coordination with the District Landscape Architect.

110.5 Control of Noxious Weeds - Exotic and Invasive Species

Highway corridors provide the opportunity for the transportation of exotic and invasive weed species through the landscape. Species that have the ability to harm the environment, human health or the economy are of particular concern. In response to the impact of exotic and invasive species, Executive Order 13112 was signed, which directs Federal Agencies to expand and coordinate efforts to combat the introduction and spread of non-native plants and animals. Grading, excavation, and fill operations during construction may introduce invasive species or promote their spreading.

Because of this, the FHWA implemented guidance for State Departments of Transportation for preventing the introduction and controlling the spread of invasive plant species on highway rights of way on transportation improvement projects. District Environmental Unit and Landscape Architecture can provide assistance in identifying invasive or exotic species which should be controlled, and in recommending mitigation or control methods to be included in appropriate highway improvement projects.

110.6 Earthquake Consideration

Earthquakes are naturally occurring events that have a high potential to cause damage and destruction. While it is not possible to completely assure earthquake proof facilities, every attempt should be made to limit potential damage and prevent collapse.

There are certain measures that should be considered when a project is to be constructed in or near a known zone of active faulting.

Early in the route location process, active and inactive faults should be mapped by engineering geologists. A general assessment of the seismic risk of various areas within the study zone should then be prepared. The DOS and Office of Structural Foundations are available to assist in the assessment of seismic risk.

Strong consideration must be given to the location of major interchanges. They must be sited outside of heavily faulted areas unless there are exceptional circumstances that make it impractical to do so. Where close seismic activity is highly probable, consideration should be given to avoiding complex multilevel interchanges in favor of simple designs with low skew, short span structures close to the original ground, and maximum use of embankment. Single span bridges which are designed to tolerate large movements are desirable.

Early recognition of seismic risk may lead the designer to modify alignment or grade in order to minimize high cuts, fills, and bridge structures in the area. Slopes should be made as flat as possible both for embankment stability and to reduce slide potential in cuts. Buttress fills can be constructed to improve cut stability. The DOS and the Office of Structural Foundations, should be consulted early when considering various alternatives to obtain recommendations for mitigating earthquake damage.

When subjected to an earthquake, fills may crack, slump, and settle. In areas of high water table, liquefaction may cause large settlement and shifting of the roadway. It is not economically feasible to entirely prevent this damage. One possible mitigation for existing soils would be to have the contract Special Provisions provide for removal of loose and compressible material from fill foundation areas, particularly in canyons, sidehill fills, and ravines and for foundation preparation on existing hillsides at the transition between cut and fill.

No modification is necessary in the design of the pavement structural sections for the purpose of reducing damage due to future earthquakes. Normally it is not possible to reduce this damage, since the structural section cannot be insulated from movements of the ground on which it rests. In active fault areas, consideration should be given to the use of flexible pipes or pipes with flexible couplings for cross drains, roadway drainage and conduits.

Additional expenditure for right of way and construction to make highways and freeways more earthquake resistant in a known active fault area should be kept in balance with the amount of impact on the traveling public if the facility may be put out of service following a disastrous earthquake. Loss of a major interchange, however, may have a tremendous influence on traffic flow and because of the secondary life-safety and economic impacts some additional expenditure may be justified.

110.7 Traffic Control Plans

This section focuses mainly on providing for vehicular traffic through the work zone; however, providing for bicyclists, pedestrians, and transit through the work zone is also necessary when they are not prohibited.

A detailed plan for moving all users of the facility through or around a construction zone must be developed and included in the PS&E for all projects to assure that adequate consideration is given to the safety and convenience of motorists, transit, bicyclists, pedestrians, and workers during construction. Design plans and specifications must be carefully analyzed in conjunction with Traffic, Construction, and Structure personnel (where applicable) to determine in detail the measures required to warn and guide motorists, transit, bicyclists, and pedestrians through the project during the various stages of work. Starting early in the design phase, the project engineer should give continuing attention to this subject, including consideration of the availability of appropriate access to the work site, in order that efficient rates of production can be maintained. In addition to reducing the time the public is exposed to construction operations, the latter effort will help to hold costs to a minimum.

The traffic control plans should be consistent with the California MUTCD, and the philosophies and requirements contained in standard traffic control system plans developed by the Headquarters Division of Traffic Operations for use on State highways and should cover, as appropriate, such items as:

- Signing.
- Flagging.
- Geometrics of detours.
- Methods and devices for delineation and channelization.
- Application and removal of pavement markings.
- Placement and design of barriers and barricades.
- Separation of opposing vehicular traffic streams (See 23 CFR 630J).
- Maximum lengths of lane closures.
- Speed limits and enforcement.
- Use of COZEEP (see Construction Manual Section 2-215).
- Use of pilot cars.
- Construction scheduling.
- Staging and sequencing.
- Length of project under construction at any one time.
- Methods of minimizing construction time without compromising safety.
- Hours of work.
- Storage of equipment and materials.
- Removal of construction debris.
- Treatment of pavement edges.
- Roadway lighting.
- Movement of construction equipment.
- Access for emergency vehicles.
- Clear roadside recovery area.
- Provision for disabled vehicles.
- Surveillance and inspection.
- Needed modifications of above items for inclement weather or darkness.
- Evaluate and provide for as appropriate the needs of bicyclists and pedestrians (including ADA requirements; see Index 105.4).
- Provisions to accommodate continued transit service.
- Consideration of complete facility closure during construction.
- Consideration of ingress/egress requirements for construction vehicles.
- Any other matters appropriate to the safety objective.

Normally, not all the above items will be pertinent to any one traffic control plan. Depending on the complexity of the project and the volume of traffic affected, the data to be included in the traffic control plan can vary from a simple graphic alignment of the various sequences to the inclusion of complete construction details in the plans and special provisions. In any event, the plans should clearly depict the exact sequence of operation, the construction details to be performed, and the traveled way to be used by all modes of traffic during each construction phase. Sufficient alignment data, profiles, plan dimensions, and typical sections should be shown to ensure that the contractor and resident engineer will have no difficulty in providing traffic-handling facilities.

In some cases, where the project includes permanent lighting, it may be helpful to install the lights as an early order of work, so they can function during construction. In other cases, temporary installations of high-level area lighting may be justified.

Temporary roadways with alignment and surfacing consistent with the standards of the road which has just been traveled by the motorist should be provided if physically and economically possible.

Based on assessments of safety benefits, relative risks and cost-effectiveness, consideration should be given to the possibility of including a bid item for continuous traffic surveillance and control during particular periods, such as:

(a) When construction operations are not in progress.

(b) When lane closures longer than a specified length are delineated by cones or other such nonpermanent devices, whether or not construction operations are in progress.

(c) Under other conditions where the risk and consequences of traffic control device failure are deemed sufficient.

Potentially hazardous working conditions must be recognized and full consideration given to the safety of workers as well as the general public during construction. This requirement includes the provision of adequate clearance between public traffic and work areas, work periods, and lane closures based on careful consideration of anticipated vehicle traffic volumes, and minimum exposure time of workers through simplified design and methods.

If a Transportation Management Plan (TMP) is included in the project, the traffic control plans (TCP) may need to be coordinated with the public information campaign and the transportation demand management elements. Any changes in TMP or TCP must be made in harmony for the plans to succeed. The “TMP Guidelines”, available from the Headquarters Division of Traffic Operations should be reviewed for further guidance.

Traffic control plans along with other features of the design should be reviewed by the District Safety Review Committee prior to PS&E as discussed in Index 110.8.

The cost of implementing traffic control plans must be included in the project cost estimate, either as one or more separate pay items or as extra work to be paid by force account.

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 be a cost-effective alternate, primarily because of the availability of good mineral aggregate in most areas of California. However, if this is a feasible option, because of unique project conditions or the potential lack of readily available materials, it may be included in a cost comparison of alternate solutions.

(2) *Flexible Pavement.* Recycling of existing flexible pavement must be considered, in all cases, as an alternative to placing 100 percent new flexible pavement.

(3) *Use of Flexible Pavement Grindings, Chunks and Pieces.* When constructing transportation facilities, the Department frequently uses asphalt in mixed or combined materials such as flexible pavement. The Department also uses recycled flexible grindings and chunks. There is a potential for these materials to reach the waters of the State through erosion or inappropriate placement during construction. Section 5650 of the Fish and Game Code states that it is unlawful to deposit asphalt, other petroleum products, or any material deleterious to fish, plant life, or bird life where they can pass into the waters of the State. In addition, Section 1601 of the Fish and Game Code requires notification to the California Department of Fish and Game (DFG) prior to construction of a project that will result in the disposal or deposition of debris, waste, or other material containing crumbled, flaked, or ground pavement where it can pass into any river, stream, or lake designated by the DFG.

The first step is to determine whether there are waters of the State in proximity to the project that could be affected by the reuse of flexible pavement. Waters of the State include: (1) perennial rivers, streams, or lakes that flow or contain water continuously for all or most of the year; or (2) intermittent lakes that contain water from time to time or intermittent rivers or streams that flow from time to time, stopping and starting at intervals, and may disappear and reappear. Ephemeral streams, which are generally exempt under provisions developed by the Department and DFG, are those that flow only in direct response to rainfall.

The reuse of flexible pavement grindings will normally be consistent with the Fish and Game Code and not require a 1601 Agreement when these materials are placed where they cannot enter the waters of the State. However, there are no set rules as to distances and circumstances applicable to the placement of asphaltic materials adjacent to waters of the State. Placement decisions must be made on case-by-case basis, so that such materials will be placed far enough away from the waters of the State to prevent weather (erosion) or maintenance operations from dislodging the material into State waters. Site-specific factors (i.e., steep slopes) should be given special care. Generally, when flexible pavement grindings are being considered for placement where there is a potential for this material to enter a water body, DFG should be notified to assist in determining whether a 1601 Agreement is appropriate. DFG may require mitigation strategies to prevent the materials from entering the Waters of the State. When in doubt, it is recommended that the DFG be notified.

If there is the potential for reused flexible materials to reach waters of the State through erosion or other means during construction, such work would normally require a 1601 Agreement. Depending on the circumstances, the following mitigation measures should be taken to prevent flexible grindings from entering water bodies:

- The reuse of flexible pavement grindings as fill material and shoulder backing must conform to the California Department of Transportation (Department) Standard Specifications, applicable manuals of instruction, contract provisions, and the MOU described below.
- Flexible chunks and pieces in embankment must be placed above the water table and covered by at least one foot of material.

A Memorandum of Understanding (MOU) dated January 12, 1993, outlines the interim agreement between the DFG and the Department regarding the use of asphaltic materials. This MOU provides a working agreement to facilitate the Department's continued use of asphaltic materials and avoid potential conflicts with the Fish and Game Code by describing conditions where use of asphalt road construction material by the Department would not conflict with the Fish and Game Code.

Specific Understandings contained in the MOU are:

- Asphalt Use in Embankments

  The Department may use flexible pavement chunks and pieces in embankments when these materials are placed where they will not enter the waters of the State.

- Use of flexible pavement grindings as Shoulder Backing

  The Department may use flexible pavement grindings as shoulder backing when these materials are placed where they will not enter the waters of the State.

- Streambed Alteration Agreements

  The Department will notify the DFG pursuant to Section 1601 of the Fish and Game Code when a project involving the use of asphaltic materials or crumbled, flaked, or ground pavement will alter or result in the deposition of pavement material into a river, stream, or lake designated by the DFG. When the proposed activity incorporates the agreements reached under Section 1601 of the Fish and Game Code, and is consistent with Section 5650 of the Fish and Game Code and this MOU, the DFG will agree to the use of these materials.

There may be circumstances where agreement between the DFG and the Department cannot be reached. Should the two agencies reach an impasse, the agencies enter into a binding arbitration process outlined in Section 1601 of the Fish and Game Code. However, keep in mind that this arbitration process does not exempt the Department from complying with the provisions of the Fish and Game Code. Also it should be noted that this process is time consuming, requiring as much as 72 days or more to complete. Negotiations over the placement of flexible pavement grindings, chunks, and pieces are to take place at the District level as part of the 1601 Agreement process.

110.12 Tunnel Safety Orders

Projects and work activities that include human entry into tunnels, shafts or any of a variety of underground structures to conduct construction activities must address the requirements of the California Code of Regulations (CCR), Title 8, Subchapter 20 – Tunnel Safety Orders (TSO). Activities that can be considered of a maintenance nature, such as cleaning of sediment and debris from culverts or inspection (either condition inspection for design purposes or inspection as a part of construction close-out) of tunnels, shafts or other underground facilities are not affected by these regulations.

TSO requires the Department, as owner of the facility, to request the Department of Industrial Relations, Division of Occupational Safety and Health (Cal-OSHA), Mining and Tunneling Unit, to review and classify tunnels and shafts for the potential presence of flammable gas and vapors prior to bidding. The intent of the TSO regulations are to protect workers from possible injury due to exposure to hazardous conditions. Failure to comply is punishable by fine. The complete TSO regulations are available at the following website: (http://www.dir.ca.gov/title8/sub20.html), with Sections 8403 and 8422 containing information most applicable to project design.

The TSO regulations require classification whenever there is human entry into a facility defined as a tunnel or entry into, or very near the entrance of, a shaft. Some of the common types of activities where human entry is likely and that will typically require classification include:

- Pipe jacking or boring operations
- Culvert rehabilitation
- Large diameter pile construction, as described in the following text
- Pump house vaults
- Cut-and-cover operations connected to ongoing underground construction and are covered in a manner that creates conditions characteristic of underground construction
- Well construction
- Cofferdam excavations
- Deep structure footings/shafts/casings, as described in the following text

Virtually any project that will lead to construction or rehabilitation work within a pipe, caisson, pile or underground structure that is covered by soil is subject to the TSO regulations. This typically applies to underground structures of 30 inches or greater diameter or shaft excavations of 20 feet or more in depth. Since a shaft is defined as any excavation with a depth at least twice its greatest cross section, the regulations will apply to some structure footing or cofferdam excavations.

Cut and cover operations (typical of most pipe, junction structure and underground vault construction) do not fall under the TSO regulations as long as worker entry to the pipe or system (usually for grouting reinforced concrete pipe, tightening bolts on structural plate pipe, etc.) is conducted prior to covering the facility with soil. Connecting new pipe to existing buried pipe or structures does fall under the TSO regulations unless the existing pipe system is physically separated by a bulkhead to prevent entry into the buried portion. Designers must either incorporate requirements for such separation of facilities into the PS&E or they must obtain the required classification from Cal-OSHA. For any project that requires classification, specifications must be included that alert the Contractor to the specific location and classification that Cal-OSHA has provided.

The TSO regulations should be viewed as being in addition to, and not excluding, other requirements as may apply to contractor or Department personnel covered in the Construction Safety Orders (see CCR, Title 8, Subchapter 4, Article 6 at http://www.dir.ca.gov/title8/sub4.html), safety and health procedures for confined spaces (see Chapter 14 of the Caltrans Safety Manual), or any other regulations that may apply to such work.

Prior to PS&E submittal on a project that includes any work defined in CCR Section 8403, a written request must be submitted for classification to the appropriate Mining and Tunneling (M&T) Unit office. Each M&T Unit office covers specific counties as shown on Figure 110.12. Classification must be obtained individually for each separate location on a project. For emergency projects or other short lead-time work, it is recommended that the appropriate M&T Unit office be contacted as soon as possible to discuss means of obtaining classification prior to the start of construction activities.

The request must include all pertinent and necessary data to allow the M&T Unit to classify the situation. The data specified under paragraph (a) of Section 8422 (complete text of Section 8422 reprinted below) is typical of new construction projects, however for culvert rehabilitation and other type of work affecting an existing facility, not all of the indicated items are typically available or necessary for submittal. The appropriate M&T Unit office should be contacted for advice if there is any question regarding data to submit.

In many instances it may not be known during design if there will be human entry into facility types that would meet the definition of a tunnel or shaft. If there is any anticipation that such entry is likely to occur, classification should be requested. As permit acquisition is typically the responsibility of the District, it is imperative that there be close coordination between District and Structures Design staff regarding the inclusion of any facilities in the structures PS&E that could be defined as a tunnel or shaft and have potential for human entry. The following text is taken directly from Section 8422:

8422 Tunnel Classifications

(a) When the preliminary investigation of a tunnel project is conducted, the owner or agency proposing the construction of the tunnel shall submit the geological information to the Division for review and classification relative to flammable gas or vapors. The preliminary classification shall be obtained from the Division prior to bidding and in all cases prior to actual underground construction. In order to make the evaluation, the following will be required:

(1) Plans and specifications;

(2) Geological report;

(3) Test bore hole and soil analysis log along the tunnel alignment;

(4) Proximity and identity of existing utilities and abandoned underground tanks.

(5) Recommendation from owner, agency, lessee, or their agent relative to the possibility of encountering flammable gas or vapors;

(6) The Division may require additional drill hole or other geologic data prior to making gas classifications.

(b) The Division shall classify all tunnels or portions of tunnels into one of the following classifications:

(1) Nongassy, which classification shall be applied to tunnels where there is little likelihood of encountering gas during the construction of the tunnel.

(2) Potentially gassy, which classification shall be applied to tunnels where there is a possibility flammable gas or hydrocarbons will be encountered.

(3) Gassy, which classification shall be applied to tunnels where it is likely gas will be encountered or if a concentration greater than 5 percent of the LEL of:

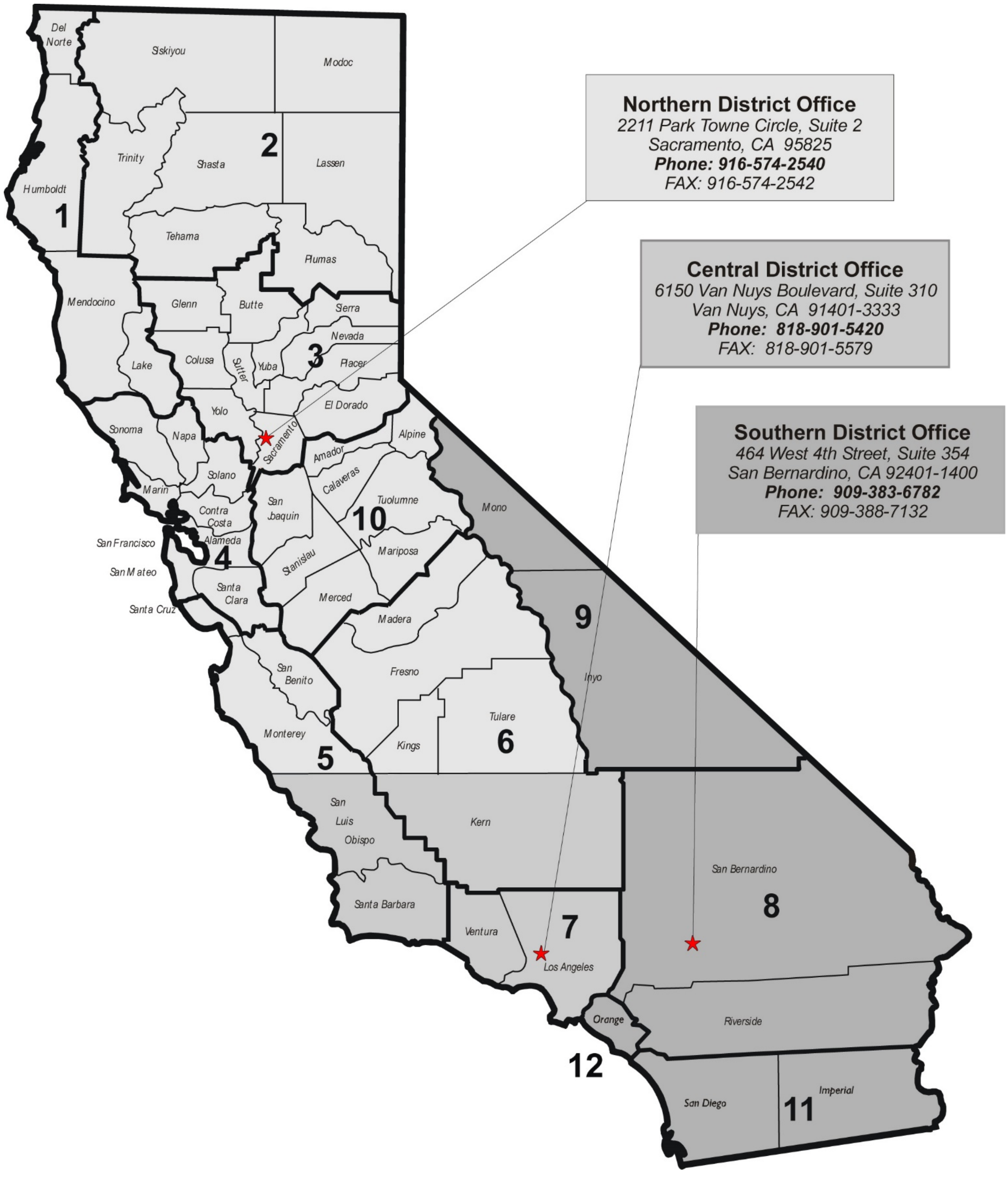
(A) flammable gas has been detected not less than 12 inches from any surface in any open workings with normal ventilation.

(B) flammable petroleum vapors that have been detected not less than 3 inches from any surface in any open workings with normal ventilation.

(4) Extrahazardous, which classification shall be applied to tunnels when the Division finds that there is a serious danger to the safety of employees and:

Flammable gas or petroleum vapor emanating from the strata has been ignited in the tunnel; or

Figure 110.12
California Mining and Tunneling Districts



(A) A concentration of 20 percent of the LEL of flammable gas has been detected not less than 12 inches from any surface in any open working with normal ventilation; or

(B) A concentration of 20 percent of LEL petroleum vapors has been detected not less than three inches from any surface in any open workings with normal ventilation.

(c) A notice of the classification and any special orders, rules, special conditions, or regulations to be used shall be prominently posted at the tunnel job site, and all personnel shall be informed of the classification.

(d) The Division shall classify or reclassify any tunnel as gassy or extrahazardous if the preliminary investigation or past experience indicates that any gas or petroleum vapors in hazardous concentrations is likely to be encountered in such tunnel or if the tunnel is connected to a gassy or extrahazardous excavation and may expose employees to a reasonable likelihood of danger.

(e) For the purpose of reclassification and to ensure a proper application of classification, the Division shall be notified immediately if a gas or petroleum vapor exceeds any one of the individual classification limits described in subsection (b) above. No underground works shall advance until reclassification has been made.

(1) A request for declassification may be submitted in writing to the Division by the employer and/or owner's designated agent whenever either of the following conditions occur:

(A) The underground excavation has been completed and/or isolated from the ventilation system and/or other excavations underway, or

(B) The identification of any specific changes and/or conditions that have occurred subsequent to the initial classification criteria such as geological information, bore hole sampling results, underground tanks or utilities, ventilation system, air quality records, and/or evidence of no intrusions of explosive gas or vapor into the underground atmosphere.

NOTE: The Division shall respond within 10 working days for any such request. Also, the Division may request additional information and/or require specific conditions in order to work under a lower level of classification.

Topic 111 - Material Sites and Disposal Sites

111.1 General Policy

The policies and procedures concerning material sites and disposal sites are listed below. For further information concerning selection and procedures for disposal, staging and borrow sites, see DIB 85.

(a) Materials investigations and environmental studies of local materials sources should be made to the extent necessary to provide a basis for study and design. Location and capacity of available disposal sites should be determined for all projects requiring disposal of more than 10,000 cubic yards of clean material. Sites for disposal of any significant amount of material in sensitive areas should be considered only where there is no practical alternative.

(b) Factual information obtained from such investigations should be made readily available to prospective bidders and contractors.

(c) The responsibility for interpreting such information rests with the contractor and not with the State.

(d) Generally, the designation of optional material sites or disposal sites will not be included in the special provisions. Mandatory sites must be designated in the special provisions or Materials Information Handout as provided in Index 111.3 of this manual and Section 2-1.03 of the Standard Specifications. A disposal site within the highway right of way (not necessarily within the project limits) should be provided when deemed in the best interest of the Department as an alternative to an approved site for disposal of water bearing residues generated by grinding or grooving operations, after approval is obtained from the Regional Water Quality Control Board (RWQCB) having jurisdiction over the area.

(e) Material agreements or other arrangements should be made with owners of material sites whenever the absence of such arrangements would result in restriction of competition in bidding, or in other instances where it is in the State's interest that such arrangements be made.

(f) The general policy of Caltrans is to avoid specifying mandatory sources unless data in support of such sources shows certain and substantial savings to the State. Mandatory sources must not be specified on Federal-aid projects except under exceptional circumstances, and prior approval of the FHWA is required. Supporting data in such cases should be submitted as early as possible. This policy also applies to disposal sites.

(g) It is the policy of Caltrans to cooperate with local authorities to the greatest practicable extent in complying with environmental requirements for all projects. Any corrective measures wanted by the local authorities should be provided through the permit process. Any unusual requirements, conditions, or situations should be submitted to the Division of Design for review (see Indexes 110.2 and 110.3).

(h) The use of any materials site requires compliance with environmental laws and regulations, which is normally a part of the project environmental documentation. If the need for a site occurs after approval of the project environmental document, a separate determination of environmental requirements for the materials site may be required.

(i) If the materials site is outside the project limits and exceeds 1-acre in size, or extraction will exceed 1,000 cubic yards, it must comply with the Surface Mining and Reclamation Act of 1975 (SMARA) and be included on the current “AB 3098 List” published by the Department of Conservation before material from that site can be used on a State project. There are limited exceptions to this requirement and the District Materials Engineer should be consulted.

111.2 Investigation of Local Materials Sources

(1) Extent of Explorations. Possible sources of materials should be investigated to the extent necessary to assure that the design of each project is based on the most economical use of available materials compatible with good environmental design practices. Where it can be reasonably assumed that all required materials can be most economically obtained from commercial sources on the current “AB 3098 List”, it should be unnecessary to investigate other sites. In all other cases material sites should be investigated. Exploration of materials sources should not be restricted to those properties where the owner expresses willingness to enter into agreement with the State. Unless it is definitely known that the owner will under no circumstances permit removal of materials, the site should be considered as a possible source of local materials.

(2) Geotechnical Design Report or Materials Report. The Geotechnical Design Report or Materials Report should include complete information on all sites investigated and should discuss the quality, cost, SMARA status, and availability of materials from commercial plants on the current “AB 3098 List”. Sufficient sampling of sites must be performed to indicate the character of the material and the elevation of the ground water surface, and to determine changes in the character of the material, both laterally and vertically. Sampling must be done in such a manner that individual samples can be taken from each horizon or layer. Composite samples of two or more different types of material are unsatisfactory, as there is no assurance that the materials would be so combined if the materials source were actually used. Testing of blends of two or more types of materials is permissible, provided the test report clearly indicates the combination tested. The test report must clearly indicate the location of the sample and the depth represented. The fact that materials sites are not designated in the Special Provisions does not reduce the importance of thorough exploration and testing.

As tabulations of test data for local materials will be furnished to prospective bidders, and the test reports may be examined by bidders if they so request, it is important that only factual data be shown on the test report and that no conclusions, opinions, or interpretation of the test data be included. Under "Remarks", give only the pertinent factual information regarding the scalping, crushing, blending, or other laboratory processing performed in preparing samples for testing, and omit any comments as to suitability for any purpose. Any discussion of the quality, suitability, or quantity of material in local materials sites necessary for design purposes should be included in the Geotechnical Design Report or Materials Report, and not noted on the test reports. For any potential materials source explored or tested, all boring and test data must be furnished, including those tests which indicate unsuitable or inferior material.

Materials information to be furnished bidders may include data on a materials source previously investigated for the same project or some other project provided all of the following conditions are met:

(a) There has been no change in test procedures subsequent to the time the earlier tests were made.

(b) The materials source has not been altered by stream action, weathering, or other natural processes.

(c) The material sampled and represented by the tests has not been removed.

(d) There has been no change in SMARA status, or inclusion or exclusion on the "AB 3098 List".

It will be necessary for each District to maintain a filing system such that all preliminary test reports for potential materials sites are readily accessible. This will necessitate preparation of test reports covering all preliminary tests of materials. It will also be essential to maintain some type of materials inventory system, whereby sites in the vicinity of any project can be readily identified and the test reports can be immediately accessible. Filing only by numerical or chronological order will not be permissible.

111.3 Materials Information Furnished to Prospective Bidders

(1) Materials Information Compilation. It is the intent that all test data applicable to material sites for a project be furnished to prospective bidders. To obtain uniformity in the "handouts" furnishing this information to prospective bidders, the District Materials Unit should develop the "handout" and the following information must be included:

(a) A cover page entitled, "Materials Information", should show District, County, Route, kilometer post limits, and geographical limits. There should be a note stating where the records, from which the information was compiled, may be inspected. Also, an index, listing investigated material sites, and disposal sites, maps, test reports, tabulation sheets, SMARA status, and agreements is to be shown on the cover page.

(b) A vicinity map showing the location of investigated materials sites and disposal sites in relation to the project.

(c) A map of each material site showing the location and identification of boring or test pits.

(d) A tabulation of the test data for each material site, showing complete information on the location, depth, and processing of each sample tested, together with all test results.

(e) Copies of all options or agreements with owners of the material sites, if such arrangements have been made.

(f) Soil survey sheets or suitable terrain maps showing borings and tests along the highway alignment.

(g) A tabulation of which sites comply with environmental laws and regulations and are included on the current "AB 3098 List".

(h) Material site grading and reclamation plan and disposal site grading plans, if they have been prepared.

(i) Copies of local use permits and clearances (when they have been obtained by the State) such as environmental clearances, mining permits, Forest Service Fire Regulations, water quality control clearances, etc. If documents are of unusual length, a statement should be included that they have been obtained and are available for inspection at the District office or Sacramento Plans Counter.

Maps, test reports, and other data included in the "Materials Information" must be factual, and should not include any comments, conclusions, or opinions as to the quality, quantity, suitability, depth, or area of the materials in any material site or along the highway.

Reproducible copies of all material to be included in the "Material Information" package should be submitted to the Office Engineer.

The Office Engineer will reproduce the "Materials Information," and copies will be available to prospective bidders upon request in the same manner that plans and special provisions are furnished.

111.4 Materials Arrangements

Materials agreements or other arrangements must be made in accordance with the policy stated under Index 111.1(e).

The determination of when and where materials agreements or other arrangements are to be obtained is the responsibility of the District, see Section 8.25.00.00 of the Right of Way Manual.

The District should also determine the maximum royalty that can be paid economically on the basis of availability of competitive sources.

In preparing agreements, guaranteed quantity provisions should not be included, as the opportunity exists for possible token removal, with the result that the State would be required to pay for the guaranteed quantity even though the material would not actually be removed. Also, requirements that the State perform construction work on the owner's property, such as fences, gates, cattle guards, roads, etc., should be included only when the cost of such items and possible resulting benefits have been properly considered in the derivation of the royalty.

111.5 Procedures for Acquisition of Material Sites and Disposal Sites

These instructions establish procedures to be followed in the purchase of material sites and disposal sites when such purchase is deemed necessary by the District. The steps to be taken are listed in order as follows:

(1) General Procedure.

(a) A District report proposing and establishing the necessity for purchase of the site is required. The report should contain the following information:

- The project or projects on which the site is to be used and programming of proposed construction.
- The location and description of the property, zoning, and site restoration/reclamation proposals including necessary vicinity and site maps.
- The amount and quality of material estimated to be available in the site and amount needed for the project or projects, or amount of excess material to be disposed of and the capacity of the site or sites.
- An economic analysis using the estimated purchase price and value of land after removal of material or deposit of excess material. The total estimated savings over other possible alternatives must be clearly demonstrated. Alternatives must be shown from the standpoint of what would have to be done if the site was not purchased. Alternatives could be changes in location or grade as well as alternative sources of material.
- A statement as to whether or not the use of the site should be mandatory, with a separate statement regarding the effect for each proposed project for which mandatory use of the site is considered necessary, including complete justification for the mandatory

specification (see Index 111.6). Three copies of each map or other attachment, folded letter size, are required for mandatory sites on all Federal-aid projects.

- A statement of the type of environmental documentation.
- Other justification.

Send one copy to the Division of Design and one copy to DES Materials Engineering and Testing Services for information.

(b) If the project or projects are to have Federal aid, the District will prepare a request, with supporting environmental clearance, for FHWA approval to specify the source as mandatory. One copy of this request should be sent to the Office Engineer and one copy to Division of Design.

(c) If the estimated purchase price is over $300,000, the District should include the item in the STIP and corresponding budget.

(d) When the proposed purchase has been approved, the Project Engineer should notify the District Division of Right of Way, District Environmental Division and the District Materials Unit and request that Right of Way purchase the site (or obtain a Materials Agreement; the Materials Unit should assist in the development of the agreement) and the Environmental Division obtain environmental authorization to proceed.

(e) The District must include the cost of purchase in the proper fiscal year program and/or budget as part of the District targets.

(f) After budgeting, the District must submit an expenditure authorization to cover purchase of the site. This could be concurrent if the project is added to the budget during a fiscal year. The expenditure authorization request should be processed through the District Project Management and Administration Units and obtain District Director approval.

(g) After issuance of an expenditure authorization, the District Division of Right of Way will complete purchase of the site.

(2) *Material and Disposal Sites in Federal Lands.* The applicable sections of the Federal Highway Act of 1958 for procurement of borrow or disposal sites, Sections 107(d) and 317, are set forth in Section 8.18.02.00 of the Right of Way Manual; Section 107(d) applies to the Interstate System while Section 317 applies to other Federal-aid highways. Whenever Federal public lands are required for a material or a disposal site, and after preliminary negotiations at the local level with the Federal agency having jurisdiction, the District must submit a letter report to the FHWA. This report should observe the requirements of Index 111.5 of this manual and Section 8.18.02.03 of the Right of Way Manual.

Following submittal of the proposal by the District to the FHWA, the latter, acting on behalf of the State transmits the proposal with a favorable recommendation to the Federal agency having control of the site. See Section 8.18.02.03 of the Right of Way Manual.

111.6 Mandatory Material Sites and Disposal Sites on Federal-aid Projects

The contract provisions must not specify a mandatory site for the disposal of surplus excavated materials unless a particular site is needed for environmental reasons or the site is found to be the most economical for one or more Federal-aid projects. All points listed in Index 111.5(1)(a) and (b) must be covered and one copy of all attachments submitted. Supporting data must be submitted to the FHWA during the project planning phase or early in the project design phase as almost all cases of mandatory sites must go to the FHWA for decision.

Section 635.407 of 23 CFR 635D states in part:

"The designation of a mandatory material source may be permitted based on environmental considerations, provided the environment would be substantially enhanced without excessive cost."

"The contract provisions ... shall not specify mandatory a site for the disposal of surplus excavated materials unless there is a finding by the State highway agency with the concurrence of the FHWA Division Administrator that such placement is the most economical except that the designation of a mandatory site may be permitted based on environmental considerations, provided the environment would be substantially enhanced without excessive cost."

Topic 112 - Contractor's Yard and Plant Sites

112.1 Policy

The Project Engineer should, during the design phase of a project, consider the need and availability of sites for the contractor's yards and materials plants. This is particularly important in areas where dust, noise, and access problems could limit the contractor in obtaining sites on their own in a timely manner. Asphalt concrete recycling projects pose special problems of material storage, access, and plant location; see Index 110.11. Temporary storage areas should be considered for grooving and grinding projects. As a general rule, the use of material sites designated in the Special Provisions should be optional. Should the materials site be desired, the contractor shall provide notice to the Resident Engineer within a designated time period after approval of the contract (30 days would be a minimum, but not more than 60 days except in unusual situations). All environmental requirements must be satisfied and local permits must be obtained prior to submittal of the PS&E. Right of Way, Permits, and Environmental units must be informed early in the process. The contractor will be allowed to use these sites only for work on the designated project(s).

112.2 Locating a Site

The Project Engineer should consult with District Division of Right of Way concerning appropriately sized parcels currently being held in the airspace inventory, nearby property held by Caltrans for future construction, or as excess land. If such space is available in the vicinity of the project, the District Environmental Division should be consulted to determine what environmental requirements are necessary for the use of these properties for the intended purpose. If sufficient space does not appear to be available for yard or plant, the Project Engineer must see that the appropriate wording is placed in the contract Special Provisions.

Topic 113 - Geotechnical Design Report

113.1 Policy

The Project Engineer must review the project initiation document and Preliminary Geotechnical Design Report, if any, to ascertain the scope of geotechnical involvement for a project. A Geotechnical Design Report (GDR) is to be prepared by the Roadway Geotechnical Engineering Branches of the Division of Engineering Services, Geotechnical Services (DES-GS) (or prepared by a consultant with technical oversight by DES-GS) for all projects that involve designs for cut slopes, embankments, earthwork, landslide remediation, retaining walls, groundwater studies, erosion control features, subexcavation and any other studies involving geotechnical investigations and engineering geology. A GDR is not required for projects that solely include those design features described in Index 114.1.

113.2 Content

The GDR is to conform to the “Guidelines for Geotechnical Reports” which is prepared by the Office of Structural Foundations.

113.3 Submittal and Review

Final copies of the GDR are to be submitted to the Project Engineer, District Materials Unit, and the Division of Design. For consultant developed reports, the GDR is to be submitted to DES-GS for review and approval. DES-GS will then transmit the approved GDR to the Project Engineer, District Materials Unit, and the Division of Design.

Topic 114 - Materials Report

114.1 Policy

A Materials Report must be prepared for all projects that involve any of the following components:

- Pavement structure recommendations and/or pavement studies
- Culverts (or other drainage materials)
- Corrosion studies
- Materials disposal sites
- Side prone areas with erosive soils

The Materials Report may be either a single report or a series of reports that contains one or several of the components listed above. Materials Reports are prepared for Project Initiation Documents, Project Reports, and PS&E. Materials Report(s) are signed and stamped with an engineers seal by the engineer in responsible charge for the findings and recommendations. The District Materials Engineer will either prepare the Materials Report or review and accept Materials Report(s) prepared by others. The Material Report is signed by the Registered Engineer that prepared the report.

114.2 Requesting Materials Report(s)

The Project Engineer (or equivalent) is responsible for requesting a Materials Report. The District Materials Engineer can assist the Project Engineer in identifying what components need to be addressed, when to request them, and what information is needed. At a minimum, the following information needs to be included in all requests:

(1) *Project location.*

(2) *Scope of work.* Project Engineer should spell out the type of work to be done that will affect materials. If pavements are involved, state type of pavement work. Provide type of project, such as new construction, widening, or rehabilitation. Note if culverts will be installed, extended, or replaced. Note if material or disposal sites are needed, see Topic 111 for criteria.

(3) *Proposed design life for pavements and culverts.*

(4) *Design Designation.* Include for projects involving pavement structural enhancements. Does not apply to pavement preservation activities.

(5) *Special Considerations or Limitations.* Include any information that may affect the materials recommendations. Examples include traffic management requirements or environmental restrictions.

114.3 Content

All Materials Reports must contain the location of the project, scope of work, and list of special conditions and assumptions used to develop the report. Materials Reports must contain the following information when the applicable activity is included in the scope of the project.

(1) *Pavement.* The Materials Report must document the design designation and climate zone or climate data used to prepare the report and recommendations. Document studies, tests, and cores performed to collect data for the report. Include deflection studies for flexible pavement rehabilitation projects (see Index 635.1). Also include pavement structure recommendations. The report should also outline special material requirements that should be incorporated such as justifications for using (or not using) particular materials in the pavement structure.

(2) *Drainage Culverts or Other Materials.* The Materials Report must contain a sufficient number of alternatives that materially meet or exceed the culvert design life (and other drainage related) standards for the Project Engineer to establish the most maintainable, constructable, and cost effective alternative in conformance with FHWA regulations (23 CFR 635D).

(3) *Corrosion.* Corrosion studies are necessary when new culverts, culvert rehabilitation, or culvert extensions are part of the scope of the project. Studies should satisfy the requirements of the “Corrosion Guidelines”. Copies of the guidelines can be obtained from the Corrosion Technology Branch in DES

Materials Engineering and Testing Services or on the DES Materials Engineering and Testing Services website.

(4) *Materials or Disposal Sites.* See Topic 111 "Material and Disposal Sites" for conditions when sites need to be identified and how to document.

114.4 Preliminary Materials Report

Because resources and/or time are sometimes limited, it is not always possible to complete all the tests and studies necessary for a final Materials Report during the planning/scooping phase. In these instances, a Preliminary Materials Report may be issued using the best information available and good engineering judgment. Accurate traffic projections and design designations are still required for the Preliminary Materials Report. Preliminary Materials Reports should not be used for project reports or PS&E development. When used, Preliminary Materials Reports must document the sources of information used and assumptions made. It must clearly state that the Preliminary Materials Report is to be used for planning and initial cost estimating only and not for final design. The Department Pavement website contains supplemental guidance for developing preliminary pavement structures.

114.5 Review and Retention of Records

A copy of the Draft Materials Report is to be submitted for review and comment to the District Materials Engineer. The District Materials Engineer reviews the document for the Department to assure that it meets the standards, policies, and other requirements found in Department manuals, and supplemental district guidance (Index 604.2(2)). If it is found that the document meets these standards, the District Materials Engineer accepts the Materials Report. If not, the report is returned with comments to the submitter.

After resolution of the comments, a final copy of the Materials Report is submitted to the District Materials Engineer who then furnishes it to the Project Engineer. The original copy of the Materials Report must be permanently retained in the District's project history file and be accessible for review by others when requested.

Topic 115 - Designing for Bicycle Traffic

115.1 General

Under the California Vehicle Code, bicyclists generally have the same rights and duties that motor vehicle drivers do when using the State highway system. For example, they make the same merging and turning movements, they need adequate sight distance, they need access to all destinations, etc. Therefore, designing for bicycle traffic and designing for motor vehicle traffic are similar and based on the same fundamental transportation engineering principles. The main differences between bicycle and motor vehicle operations are lower speed and acceleration capabilities, as well as greater sensitivity to out of direction travel and steep uphill grades. Design guidance that addresses the safety and mobility needs of bicyclists on Class II bikeways (bike lanes) is distributed throughout this manual. See Chapter 1000 for additional bicycle guidance for Class I bikeways (bike paths) and Class III bikeways (bike routes). See Design Information Bulletin (DIB) 89 for Class IV bikeways (separated bikeways) guidance.

All city, county, regional and other local agencies responsible for bikeways or roads except those freeway segments where bicycle travel is prohibited shall equal or exceed the minimum bicycle design criteria contained in this and other chapters of this manual (see the Streets and Highways Code, Section 891). The decision to develop bikeways should be made in consultation and coordination with local agencies responsible for bikeway planning to ensure connectivity and network development.

Generally speaking, bicycle travel can be enhanced by bikeways or improvements to the right-hand portion of roadways, where bicycles are required to travel. When feasible, a wider shoulder than minimum standard should be considered since bicyclists are required to ride to as far to the right as possible, and shoulders provide bicyclists an opportunity to pull over to let faster traffic pass.

All transportation improvements are an opportunity to improve safety, access, and mobility for the bicycle mode of travel.

Topic 116 - Bicyclists and Pedestrians on Freeways

116.1 General

Seldom is a freeway shoulder open to bicycle, pedestrian or other non-motorized travel, but they can be opened for use if certain criteria assessing the safety and convenience of the freeway, as compared with available alternate routes, is met. However, a freeway should not be opened to bicycle or pedestrian use if it is determined to be incompatible. The Headquarters Traffic Liaison and the Project Delivery Coordinator must approve any proposals to open freeways to bicyclists, pedestrian or other non-motorized use. See the California MUTCD and CVC Section 21960.

When a new freeway segment is to remain open or existing freeway segment is to be reopened to these modes, it is necessary to evaluate the freeway features for their compatibility with safe and efficient travel, including:

- Shoulder widths
- Drainage grates; see Index 1003.5(2)
- Expansion joints
- Utility access covers on shoulders
- Frequency and spacing of entrance/exit ramps
- Multiple-lane entrance/exit ramps
- Traffic volumes on entrance/exit ramps and on lanes merging into exit ramps
- Sight distance at entrance/exit ramps
- Freeway to freeway interchanges
- The presence and design of rumble strips
- Longitudinal edges and joints

If a freeway segment has no suitable non-freeway alternative and is closed because certain features are considered incompatible, the feasibility of eliminating or reducing the incompatible features should be evaluated. This evaluation may include removal, redesign, replacement, relocation or retrofitting of the incompatible feature, or installation of signing, pavement markings, or other traffic control devices.

Where no reasonable, convenient and safe non-freeway alternative exists within a freeway corridor, the Department should coordinate with local agencies to develop new routes, improve existing routes or provide parallel bicycle and pedestrian facilities within or adjacent to the freeway right of way. See Project Development Procedures Manual Chapter 1, Article 3 (Regional and System Planning) and Chapter 31 (Nonmotorized Transportation Facilities) for discussion of the development of non-freeway transportation alternatives.

CHAPTER 200 GEOMETRIC DESIGN AND STRUCTURE STANDARDS

Topic 201 - Sight Distance

Index 201.1 - General

Sight distance is the continuous length of highway ahead, visible to the highway user. Four types of sight distance are considered herein: passing, stopping, decision, and corner. Passing sight distance is used where use of an opposing lane can provide passing opportunities (see Index 201.2). Stopping sight distance is the minimum sight distance for a given design speed to be provided on multilane highways and on 2-lane roads when passing sight distance is not economically obtainable. Stopping sight distance also is to be provided for all users, including motorists and bicyclists, at all elements of interchanges and intersections at grade, including private road connections (see Topic 504, Index 405.1, & Figure 405.7). Decision sight distance is used at major decision points (see Indexes 201.7 and 504.2). Corner sight distance is used at intersections (see Index 405.1, Figure 405.7, and Figure 504.3J).

Table 201.1 shows the minimum standards for stopping sight distance related to design speed for motorists. Stopping sight distances given in the table are suitable for Class II and Class III bikeways. The stopping sight distances are also applicable to roundabout design on the approach roadway, within the circulatory roadway, and on the exits prior to the pedestrian crossings. Also shown in Table 201.1 are the values for use in providing passing sight distance.

See Chapter 1000 for Class I bikeway sight distance guidance.

Chapter 3 of "A Policy on Geometric Design of Highways and Streets," AASHTO, contains a thorough discussion of the derivation of stopping sight distance.

201.2 Passing Sight Distance

Passing sight distance is the minimum sight distance required for the driver of one vehicle to pass another vehicle safely and comfortably. Passing must be accomplished assuming an oncoming vehicle comes into view and maintains the design speed, without reduction, after the overtaking maneuver is started.

Table 201.1
Sight Distance Standards

Design Speed(1) (mph)	Stopping(2) (ft)	Passing (ft)
10	50	---
15	100	---
20	125	800
25	150	950
30	200	1,100
35	250	1,300
40	300	1,500
45	360	1,650
50	430	1,800
55	500	1,950
60	580	2,100
65	660	2,300
70	750	2,500
75	840	2,600
80	930	2,700

(1) See Topic 101 for selection of design speed.
(2) For sustained downgrades, refer to advisory standard in Index 201.3

The sight distance available for passing at any place is the longest distance at which a driver whose eyes are 3 ½ feet above the pavement surface can see the top of an object 4 ¼ feet high on the road. See Table 201.1 for the calculated values that are associated with various design speeds.

In general, 2-lane highways should be designed to provide for passing where possible, especially those routes with high volumes of trucks or recreational vehicles. Passing should be done on tangent horizontal alignments with constant grades or a slight sag vertical curve. Not only are drivers reluctant to pass on a long crest vertical curve, but it is impracticable to design crest vertical curves to provide for passing sight distance because of high cost where crest cuts are involved. Passing sight distance for crest vertical curves is 7 to 17 times longer than the stopping sight distance.

Ordinarily, passing sight distance is provided at locations where combinations of alignment and profile do not require the use of crest vertical curves.

Passing sight distance is considered only on 2-lane roads. At critical locations, a stretch of 3- or 4-lane passing section with stopping sight distance is sometimes more economical than two lanes with passing sight distance.

Passing on sag vertical curves can be accomplished both day and night because headlights can be seen through the entire curve.

See Part 3 of the California Manual on Uniform Traffic Control Devices (California MUTCD) for criteria relating to the placement of barrier striping for no-passing zones. Note, that the passing sight distances shown in the California MUTCD are based on traffic operational criteria. Traffic operational criteria are different from the design characteristics used to develop the values provided in Table 201.1 and Chapter 3 of AASHTO, A Policy on Geometric Design of Highways and Streets. The aforementioned table and AASHTO reference are also used to design the vertical profile and horizontal alignment of the highway. Consult the Headquarters (HQ) Traffic Liaison when using the California MUTCD criteria for traffic operating-control needs.

Other means for providing passing opportunities, such as climbing lanes or turnouts, are discussed in Index 204.5. Chapter 3 of AASHTO, A Policy on Geometric Design of Highways and Streets, contains a thorough discussion of the derivation of passing sight distance.

201.3 Stopping Sight Distance

The minimum stopping sight distance is the distance required by the user, traveling at a given speed, to bring the vehicle or bicycle to a stop after an object ½-foot high on the road becomes visible. Stopping sight distance for motorists is measured from the driver's eyes, which are assumed to be 3 ½ feet above the pavement surface, to an object ½-foot high on the road. See Index 1003.1(10) for Class I bikeway stopping sight distance guidance.

The stopping sight distances in Table 201.1 should be increased by 20 percent on sustained downgrades steeper than 3 percent and longer than one mile.

201.4 Stopping Sight Distance at Grade Crests

Figure 201.4 shows graphically the relationships between length of highway crest vertical curve, design speed, and algebraic difference in grades. Any one factor can be determined when the other two are known.

201.5 Stopping Sight Distance at Grade Sags

From the curves in Figure 201.5, the minimum length of vertical curve which provides headlight sight distance in grade sags for a given design speed can be obtained.

If headlight sight distance is not obtainable at grade sags, lighting may be considered. The District approval authority or Project Delivery Coordinator, depending upon the current District Design Delegation Agreement, and the HQ Traffic Liaison shall be contacted to review proposed grade sag lighting to determine if such use is appropriate.

201.6 Stopping Sight Distance on Horizontal Curves

Where an object off the pavement such as a bridge pier, building, cut slope, or natural growth restricts sight distance, the minimum radius of curvature is determined by the stopping sight distance.

Available stopping sight distance on horizontal curves is obtained from Figure 201.6. It is assumed that the driver's eye is 3 ½ feet above the center of the inside lane (inside with respect to curve) and the object is ½-foot high. The line of sight is assumed to intercept the view obstruction at the midpoint of the sight line and 2 feet above the center of the inside lane when the road profile is flat (i.e. no vertical curve). Crest vertical curves can cause additional reductions in sight distance. The clear distance (*m*) is measured from the center of the inside lane to the obstruction.

The design objective is to determine the required clear distance from centerline of inside lane to a retaining wall, bridge pier, abutment, cut slope, or other obstruction for a given design speed. Using radius of curvature and minimum sight distance for that design speed, Figure 201.6 gives the clear distance (*m*) from centerline of inside lane to the obstruction.

See Index 1003.1(12) for bikeway stopping sight distance on horizontal curve guidance.

When the radius of curvature and the clear distance to a fixed obstruction are known, Figure 201.6 also gives the sight distance for these conditions.

See Index 101.1 for technical reductions in design speed caused by partial or momentary horizontal sight distance restrictions. See Index 203.2 for additional comments on glare screens.

Cuts may be widened where vegetation restricting horizontal sight distance is expected to grow on finished slopes. Widening is an economic trade-off that must be evaluated along with other options. See Index 902.2 for sight distance requirements on landscape projects.

201.7 Decision Sight Distance

At certain locations, sight distance greater than stopping sight distance is desirable to allow drivers time for decisions without making last minute erratic maneuvers (see Chapter III of AASHTO, A Policy on Geometric Design of Highways and Streets, for a thorough discussion of the derivation of decision sight distance.)

<u>On freeways and expressways the decision sight distance values in Table 201.7 should be used at lane drops and at off-ramp noses to interchanges, branch connections, roadside rests, vista points, and inspection stations</u>. When determining decision sight distance on horizontal and vertical curves, Figures 201.4, 201.5, and 201.6 can be used. Figure 201.7 is an expanded version of Figure 201.4 and gives the relationship among length of crest vertical curve, design speed, and algebraic difference in grades for much longer vertical curves than Figure 201.4.

Decision sight distance is measured using the 3 ½-foot eye height and ½-foot object height. See Index 504.2 for sight distance at secondary exits on a collector-distributor road.

Table 201.7
Decision Sight Distance

Design Speed (mph)	Decision Sight Distance (ft)
30	450
35	525
40	600
45	675
50	750
55	865
60	990
65	1,050
70	1,105
75	1,180
80	1,260

Topic 202 - Superelevation

202.1 Basic Criteria

According to the laws of mechanics, when a vehicle travels on a curve it is forced outward by centrifugal force.

On a superelevated highway, this force is resisted by the vehicle weight component parallel to the superelevated surface and side friction between the tires and pavement. It is impractical to balance centrifugal force by superelevation alone, because for any given curve radius a certain superelevation rate is exactly correct for only one driving speed. At all other speeds there will be a side thrust either outward or inward, relative to the curve center, which must be offset by side friction.

If the vehicle is not skidding, these forces are in equilibrium as represented by the following equation, which is used to design a curve for a comfortable operation at a particular speed:

$$\text{Centrifugal Factor} = e + f = \frac{0.067V^2}{R} = \frac{V^2}{15R}$$

Where:

- e = Superelevation slope in feet per foot
- e_{max} = Maximum superelevation rate for a given condition
- f = Side friction factor
- R = Curve radius in feet
- V = Velocity in miles per hour

Standard superelevation rates are designed to hold the portion of the centrifugal force that must be taken up by tire friction within allowable limits. Friction factors as related to speed are shown on Figure 202.2. The factors apply equally to portland cement concrete and bituminous pavements.

202.2 Standards for Superelevation

(1) *Highways.* Maximum superelevation rates for various highway conditions are shown in Table 202.2.

Based on an e_{max} selected by the designer for one of the conditions, superelevation rates from Table 202.2 shall be used within the given range of curve radii. If less than standard superelevation rates are approved (see Index 82.1), Figure 202.2 shall be used to determine superelevation based on the curve radius and maximum comfortable speed.

Maximum comfortable speed is determined by the formula given on Figure 202.2. It represents the speed on a curve where discomfort caused by centrifugal force is evident to a driver. Side friction factors tabulated on Figure 202.2 are recommended by AASHTO for design purposes. AASHTO, A Policy on Geometric Design of Highways and Streets, states, "In general, studies show that the maximum side friction factors developed between new tires and wet concrete pavements range from about 0.5 at 20 miles per hour to approximately 0.35 at 60 miles per hour." The design side friction factors are, therefore, about one-third the values that occur when side skidding is imminent.

To use Figure 202.2, the designer must decide on the relative importance among three variables. Normally, when a nonstandard superelevation rate is approved, Figure 202.2 will be entered with the rate and a desired curve radius. It must then be determined whether the resulting maximum comfortable speed is adequate for the conditions or whether further adjustments to radius and superelevation may be needed.

Except for short radius curves, the standard superelevation rate results in very little side thrust at speeds less than 45 miles per hour. This provides maximum comfort for most drivers.

Superelevation for horizontal curves with radii of 10,000 feet and greater may be deleted in those situations where the combination of a flat grade and a superelevation transition would create undesirable drainage conditions on the pavement.

Superelevated cross slopes on curves extend the full width of the traveled way and shoulders, except that the shoulder slope on the low side should be not less than the minimum shoulder slope used on the tangents (see Index 304.3 for cross slopes under cut widening conditions).

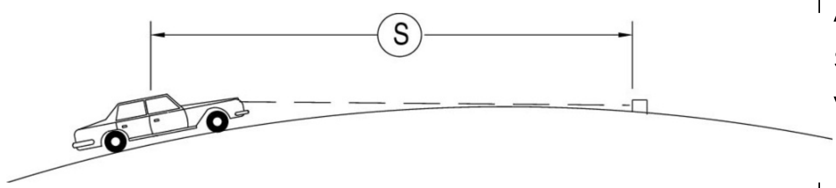
On rural 2-lane roads, superelevation should be on the same plane for the full width of traveled way and shoulders, except on transitions (see Index 304.3 for cut widening conditions).

(2) *Bikeways.* Table 202.2 also applies to Class II and III bikeways. See Index 1003.1 for Class I guidance.

202.3 Restrictive Conditions

Lower superelevation rates than those given in either Table 202.2 or Figure 202.2 may be necessary in areas where restricted speed zones or ramp/street intersections are controlling factors. Other typical locations are short radius curves on ramps near the local road juncture, either at an intersection or where a loop connects with an overcrossing structure. Often, established street grades, curbs, or drainage may prove difficult to alter and/or superelevation transition lengths would be undesirably short.

Figure 201.4
Stopping Sight Distance on Crest Vertical Curves



L = Curve Length (feet)

A = Algebraic Grade Difference (%)

S = Sight Distance (feet)

V = Design Speed for "S" in mph

K = Distance in feet required to achieve a 1% change in grade. K value as shown on graph is valid when S < L.

Drivers eye height is 3 ½ feet.
Object height is ½-foot.

Notes:

- Before using this figure for intersections, branch connections and exits, see Indexes 201.7 and 405.1, and Topic 504.
- See Figure 204.4 for vertical curve formulas.
- See Index 204.4 for minimum length of vertical curve

When S > L	**When S < L**
L = 2S – 1329/A	**L = AS^2 /1329**

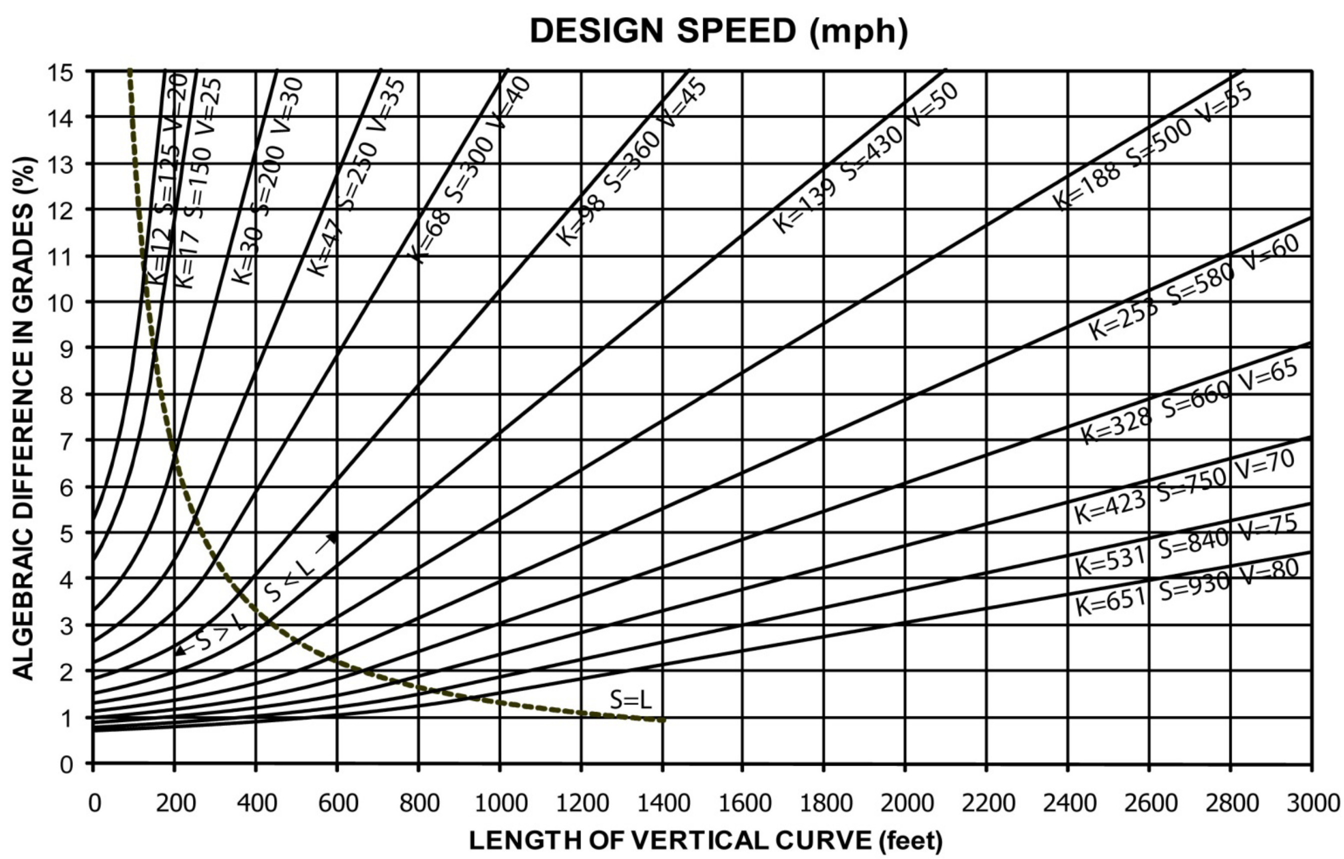
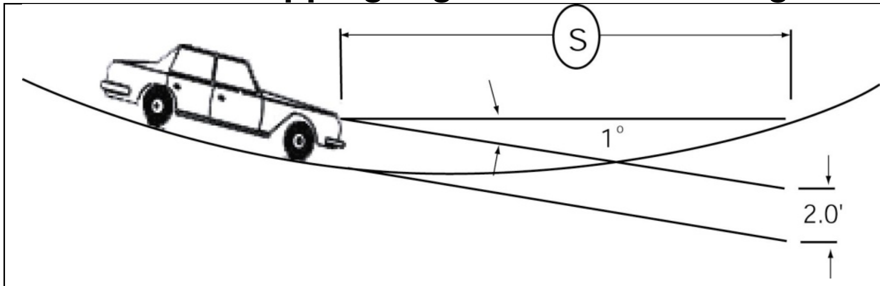


Figure 201.5
Stopping Sight Distance on Sag Vertical Curves



L = Curve Length (feet)
A = Algebraic Grade Difference (%)
S = Sight Distance (feet)
V = Design Speed for "S" in mph
K = Distance in feet required to achieve a 1% change in grade. K value as shown on graph is valid when S < L.

Notes:

- For sustained downgrades, see Index 201.3.
- Before using this figure for intersections, branch connections and exits, see Indexes 201.7 and 405.1, and Topic 504.
- See Figure 204.4 for vertical curve formulas.
- See Index 204.4 for minimum length of vertical curve.

When S > L	**When S < L**
L = 2S - (400 + 3.5S)/A	**L = AS^2 /(400 + 3.5S)**

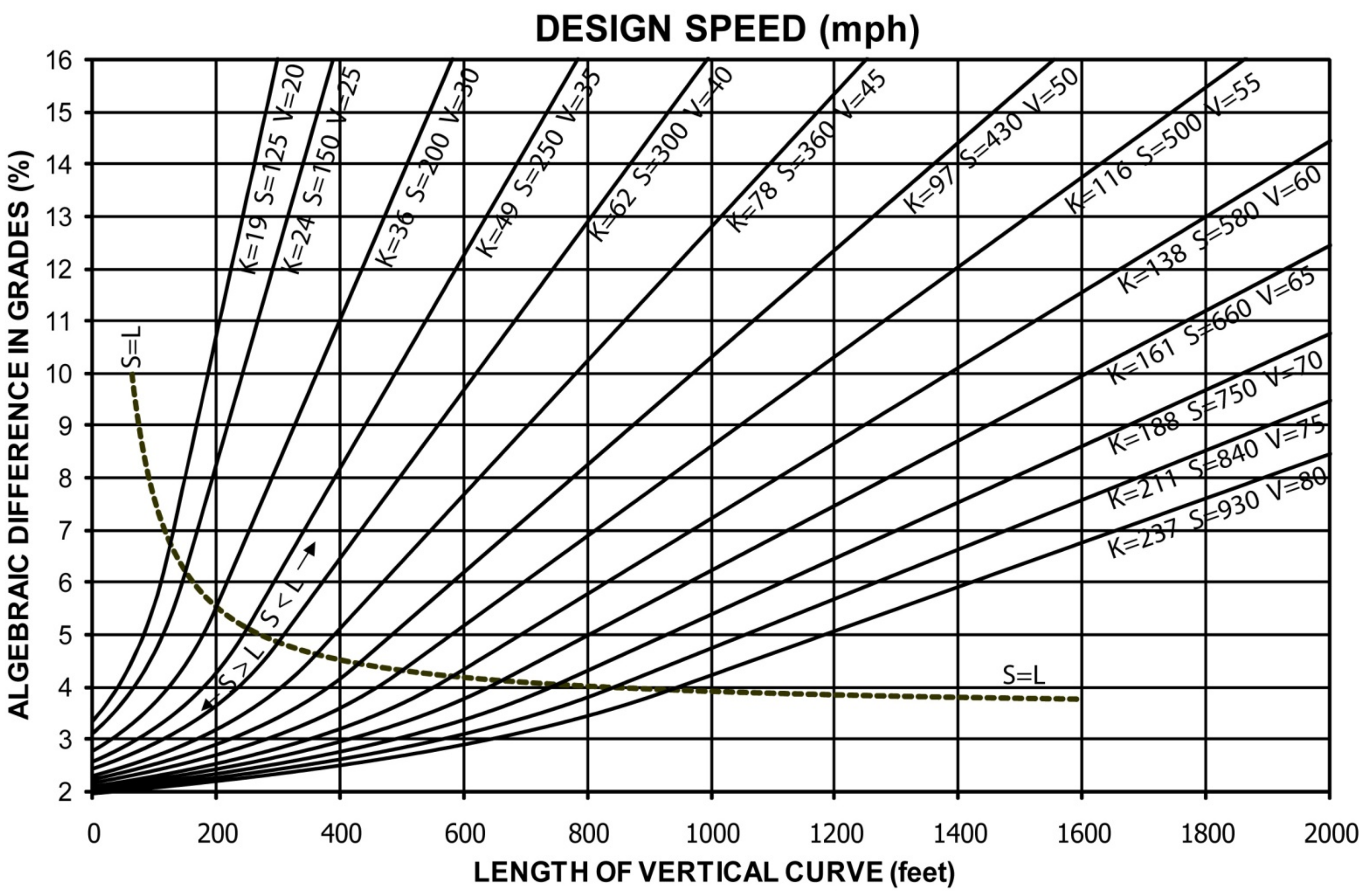
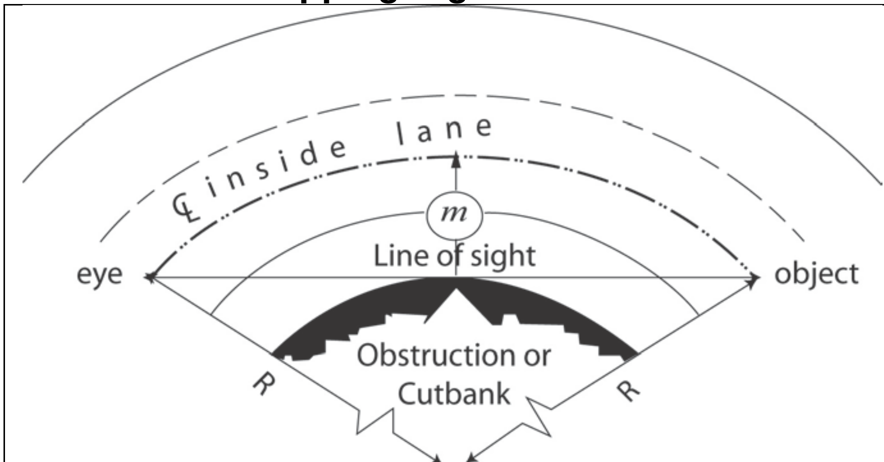


Figure 201.6
Stopping Sight Distance on Horizontal Curves



Line of sight is 2.0 feet above the centerline inside lane at point of obstruction.

R = Radius of the centerline of the lane nearest the obstruction (feet).

S = Sight Distance (feet)

V = Design Speed for "S" in mph

m = Clear distance from centerline of the lane nearest the obstruction (feet).

Notes:

- For sustained downgrades, see Index 201.3.
- Formulas apply only when "S" is equal to or less than length of curve.
- Angles in formulas are expressed in degrees.

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

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

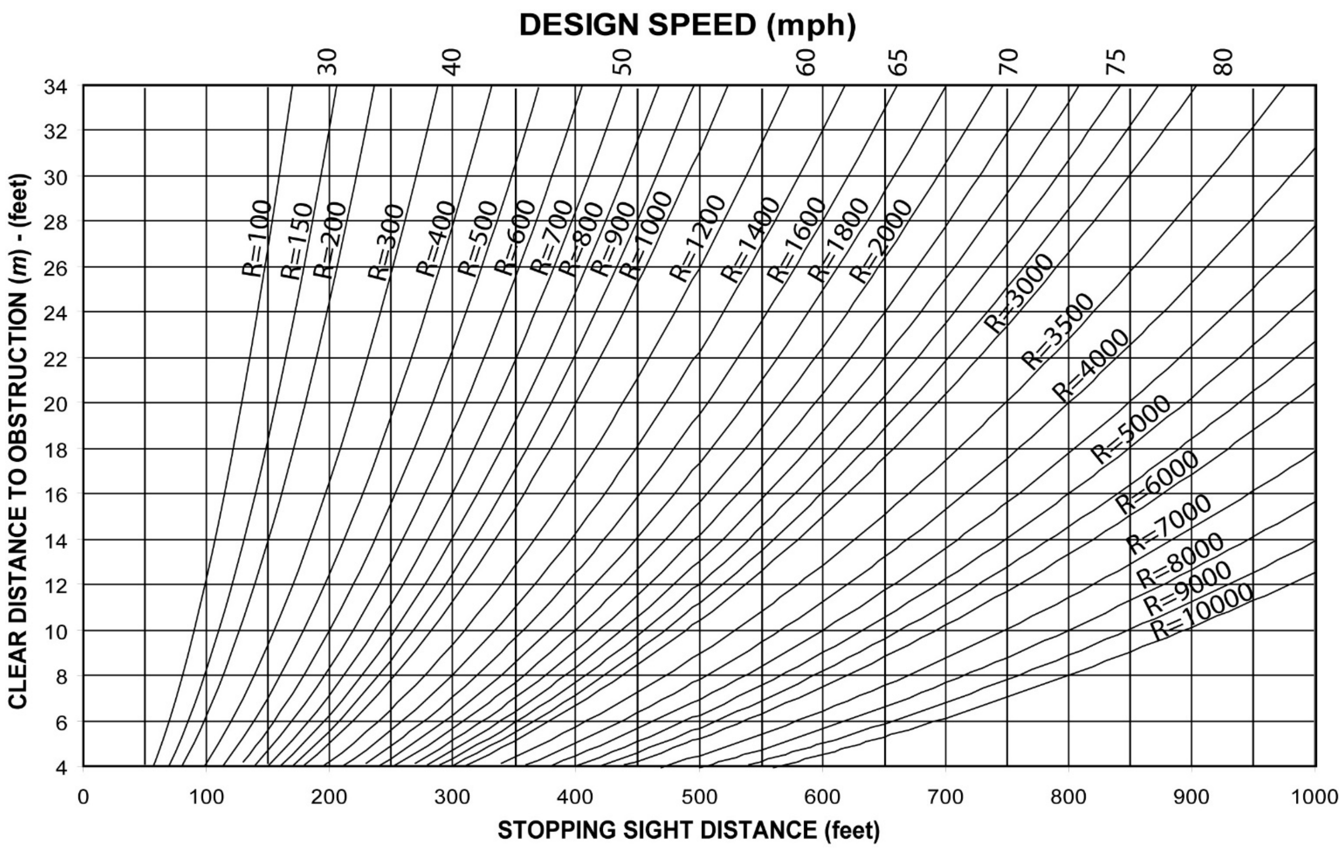
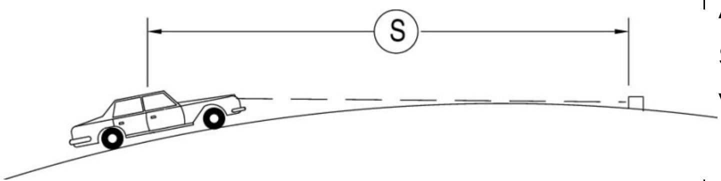


Figure 201.7
Decision Sight Distance on Crest Vertical Curves



Drivers eye height is 3½ feet.
Object height is ½-foot.

L = Curve Length (feet)

A = Algebraic Grade Difference (%)

S = Sight Distance (feet)

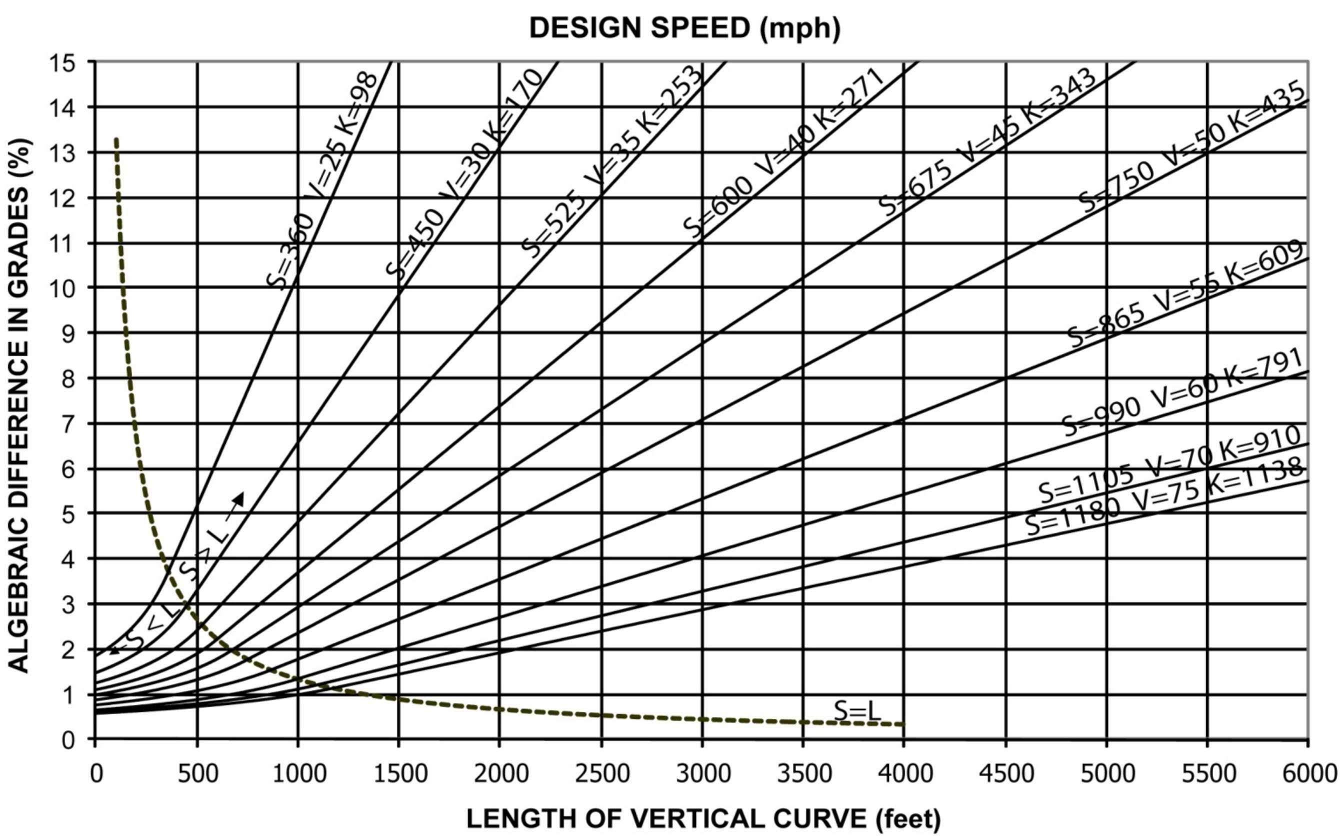
V = Design Speed for "S" in mph

K = Distance in feet required to achieve a 1% change in grade. K value as shown on graph is valid when S < L.

Notes:

- Before using this figure for intersections, branch connections and exits, see Indexes 201.7 and 405.1, and Topic 504.
- See Figure 204.4 for vertical curve formulas.
- See Index 204.4 for minimum length of vertical curve.

When S > L	**When S < L**
L = 2S – 1329/A	**L = AS^2 /1329**



Such conditions may justify a reduction in the superelevation rate, different rates for each half of the roadbed, or both. In any case, the superelevation rate provided should be appropriate for the conditions allowing for a smooth transition while providing the maximum level of comfort to the driver. Where standard superelevation rates cannot be attained, discussions should be held with the District Design Liaison and/or the Project Delivery Coordinator to determine the proper solution and the necessity of preparing a design exception fact sheet. In warping street or ramp surface areas for drainage, adverse superelevation should be avoided (see Figure 202.2).

202.4 Axis of Rotation

(1) *Undivided Highways.* For undivided highways the axis of rotation for superelevation is usually the centerline of the roadbed. However, in special cases such as desert roads where curves are preceded by long relatively level tangents, the plane of superelevation may be rotated about the inside edge of traveled way to improve perception of the curve. In flat country, drainage pockets caused by superelevation may be avoided by changing the axis of rotation from the centerline to the inside edge of traveled way.

(2) *Ramps and Freeway-to-freeway Connections.* The axis of rotation may be about either edge of traveled way or centerline if multilane. Appearance and drainage considerations should always be taken into account in selection of the axis of rotation.

(3) *Divided Highways.*

(a) Freeways--Where the initial median width is 65 feet or less, the axis of rotation should be at the centerline.

Where the initial median width is greater than 65 feet and the ultimate median width is 65 feet or less, the axis of rotation should be at the centerline, except where the resulting initial median slope would be steeper than 10:1. In the latter case, the axis of rotation should be at the ultimate median edges of traveled way.

Where the ultimate median width is greater than 65 feet, the axis of rotation should normally be at the ultimate median edges of traveled way.

To avoid sawtooth on bridges with decked medians, the axis of rotation, if not already on centerline, should be shifted to the centerline.

(b) Conventional Highways--The axis of rotation should be considered on an individual project basis and the most appropriate case for the conditions should be selected.

Aesthetics, grade distortion, superelevation transitions, drainage, and driver perception should be considered when selecting the axis of rotation (see Index 204.2).

202.5 Superelevation Transition

(1) *General.* The superelevation transition generally consists of the crown runoff and the superelevation runoff as shown on Figure 202.5A and 202.5B.

<u>A superelevation transition should be designed in accordance with the diagram and tabular data shown in Figure 202.5A to satisfy the requirements of safety, comfort and pleasing appearance</u>. The length of superelevation transition should be based upon the combination of superelevation rate and width of rotated plane in accordance with the tabulated superelevation runoff lengths on the bottom of Figure 202.5A.

Edge of traveled way and shoulder profiles should be plotted and irregularities resulting from interactions between the superelevation transition and vertical alignment of the roadway should be eliminated by introducing smooth curves. Edge of traveled way and shoulder profiles also will reveal flat areas which are undesirable from a drainage standpoint and should be avoided.

(2) *Runoff.* <u>Two-thirds of the superelevation runoff should be on the tangent and one-third within the curve</u>. This results in two-thirds of the full superelevation rate at the beginning or ending of a curve. This may be altered as required to adjust for flat spots or unsightly sags and humps, or when conforming to existing roadway.

Table 202.2

Standard Superelevation Rates
(Superelevation in Feet per Foot for Curve Radius in Feet)

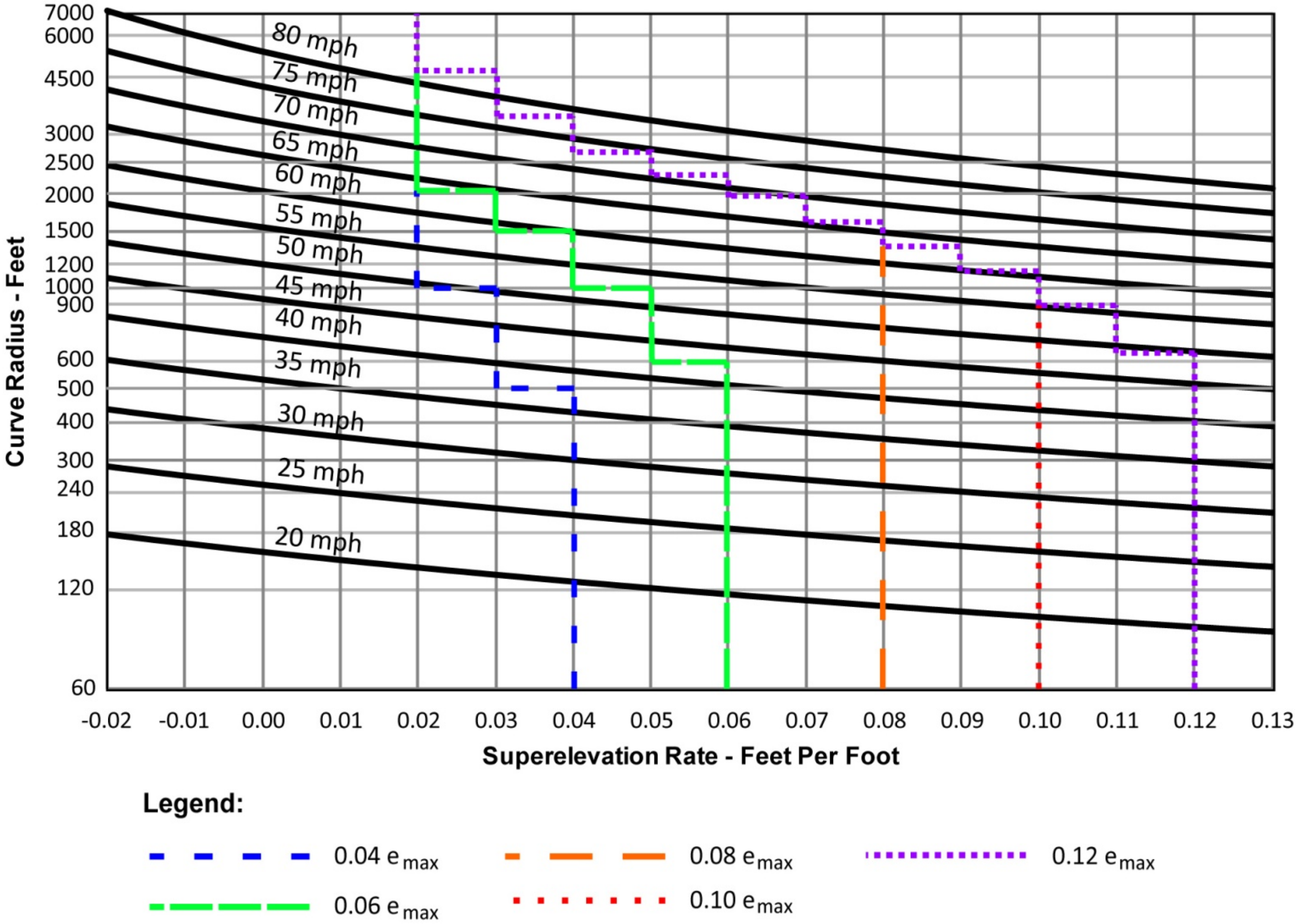
Ramps, 2-Lane Conventional Highways, Frontage Roads[1]		Freeways, Expressways, Multilane Conventional Highways		When Snow & Ice Conditions Prevail (Usually over 3,000 ft elevation)		Urban Roads (35 – 45 mph)		Urban Roads (less than 35 mph)	
For e_{max} = 0.12		For e_{max} = 0.10		For e_{max} = 0.08		For e_{max} = 0.06		For e_{max} = 0.04	
Range of Curve Radii	e Rate	Range of Curve Radii	e Rate	Range of Curve Radii	e Rate	Range of Curve Radii	e Rate	Range of Curve Radii	e Rate
Under 625	0.12								
625 – 849	0.11								
850 – 1,099	0.10	Under 1,100	0.10						
1,100 – 1,349	0.09	1,100 – 1,349	0.09						
1,350 – 1,599	0.08	1,350 – 1,599	0.08	Under 1,600	0.08				
1,600 – 1,899	0.07	1,600 – 1,899	0.07	1,600 – 1,899	0.07				
1,900 – 2,199	0.06	1,900 – 2,199	0.06	1,900 – 2,199	0.06	Under 600	0.06		
2,200 – 2,699	0.05	2,200 – 2,699	0.05	2,200 – 2,699	0.05	600 – 999	0.05		
2,700 – 3,499	0.04	2,700 – 3,499	0.04	2,700 – 3,499	0.04	1,000 – 1,499	0.04	Under 500	0.04
3,500 – 4,499	0.03	3,500 – 4,499	0.03	3,500 – 4,499	0.03	1,500 – 1,999	0.03	500 – 999	0.03
4,500 – 19,999	0.02	4,500 – 19,999	0.02	4,500 – 19,999	0.02	2,000 – 6,999	0.02	1,000 – 4,999	0.02
20,000 & over	(2)	20,000 & over	(2)	20,000 & over	(2)	7,000 & over	(2)	5,000 & over	(2)

NOTES:

(1) For frontage roads under other jurisdictions see Index 202.7.

(2) Use standard crown section.

Figure 202.2
Maximum Comfortable Speed on Horizontal Curves



Speed (mph)	Side Friction Factor "f"
20	0.17
30	0.16
40	0.15
50	0.14
55	0.13
60	0.12
65	0.11
70	0.10
75	0.09
80	0.08

NOTES:
This figure is not intended to represent standard superelevation rates or curve radius. The standards are contained in Tables 202.2 and 203.2. This figure should be used as an aid to designers to determine maximum comfortable speeds. Use of this figure in lieu of the standards must be documented as discussed in Index 82.2.

e - Superelevation
f - Side Friction Factor
V - Speed (mph)
R - Radius (feet)

$$e + f = \frac{0.067V^2}{R}$$

(3) *Restrictive Situations.* In restrictive situations, such as on two lane highways in mountainous terrain, interchange ramps, collector roads, frontage roads, etc., where curve radius and length and tangents between curves are short, standard superelevation rates and/or transitions may not be attainable. In such situations the highest possible superelevation rate(s) and transition length should be used, but the rate of change of cross slope should not exceed 6 percent per 100 feet.

(4) *Superelevation Transitions on Bridges*. Superelevation transitions on bridges should be avoided whenever possible (See Index 203.9).

(5) *Shoulder Transitions.* The shoulder plane rotates about the adjacent edge of traveled way as well as the rotational axis of the traveled way. Shoulder superelevation transitions should be smooth and compatible with the transition of the adjacent pavements.

202.6 Superelevation of Compound Curves

Superelevation of compound curves should follow the procedure as shown in Figure 202.6. Where feasible, the criteria in Index 202.5 should apply.

202.7 Superelevation on City Streets and County Roads

Superelevation rates of local streets and roads which are within the State right of way (with or without connection to State facilities) shall conform to AASHTO standards, for the functional classification of the facility in question. 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.

See Index 202.2 and Table 202.2 for Frontage Roads within the State right of way. Frontage roads that will be relinquished after construction should follow AASHTO or local standards as stated above.

Topic 203 - Horizontal Alignment

203.1 General Controls

Horizontal alignment should provide for safe and continuous operation at a uniform design speed for substantial lengths of highway. The standards which follow apply to curvature on both 2-lane and multilane highways except when otherwise noted. These standards 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.

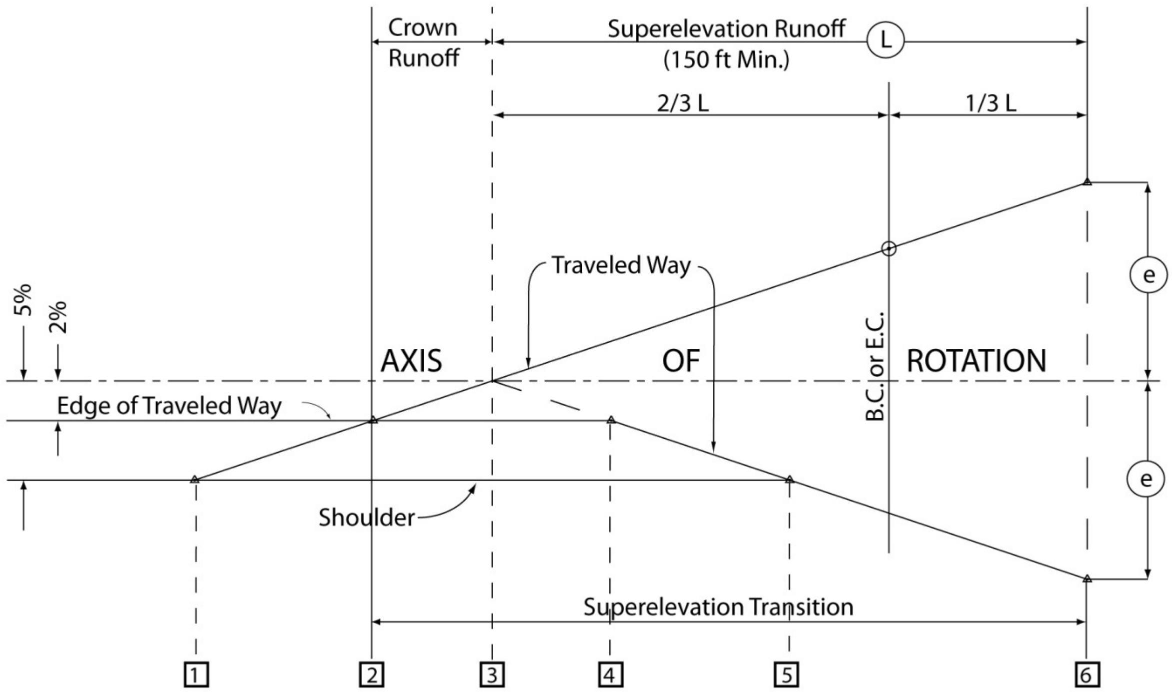
The major considerations in horizontal alignment design are safety, profile, type of facility, design speed, geotechnical features, topography, right of way cost and construction cost. In design, safety is always considered, either directly or indirectly. On freeways in metropolitan areas, alternative studies often indicate that right of way considerations influence alignment more than any other single factor. Topography controls both curve radius and design speed to a large extent. The design speed, in turn, controls sight distance, but sight distance must be considered concurrently with topography because it often demands a larger radius than the design speed. All these factors must be balanced to produce an alignment which optimizes the achievement of various objectives such as safety, cost, harmony with the natural contour of the land, and at the same time adequate for the design classification of the highway.

Horizontal alignment shall provide at least the minimum stopping sight distance for the chosen design speed at all points on the highway, as given in Table 201.1 and explained in Index 201.3. See Index 101.1 for technical reductions in design speed.

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 De	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



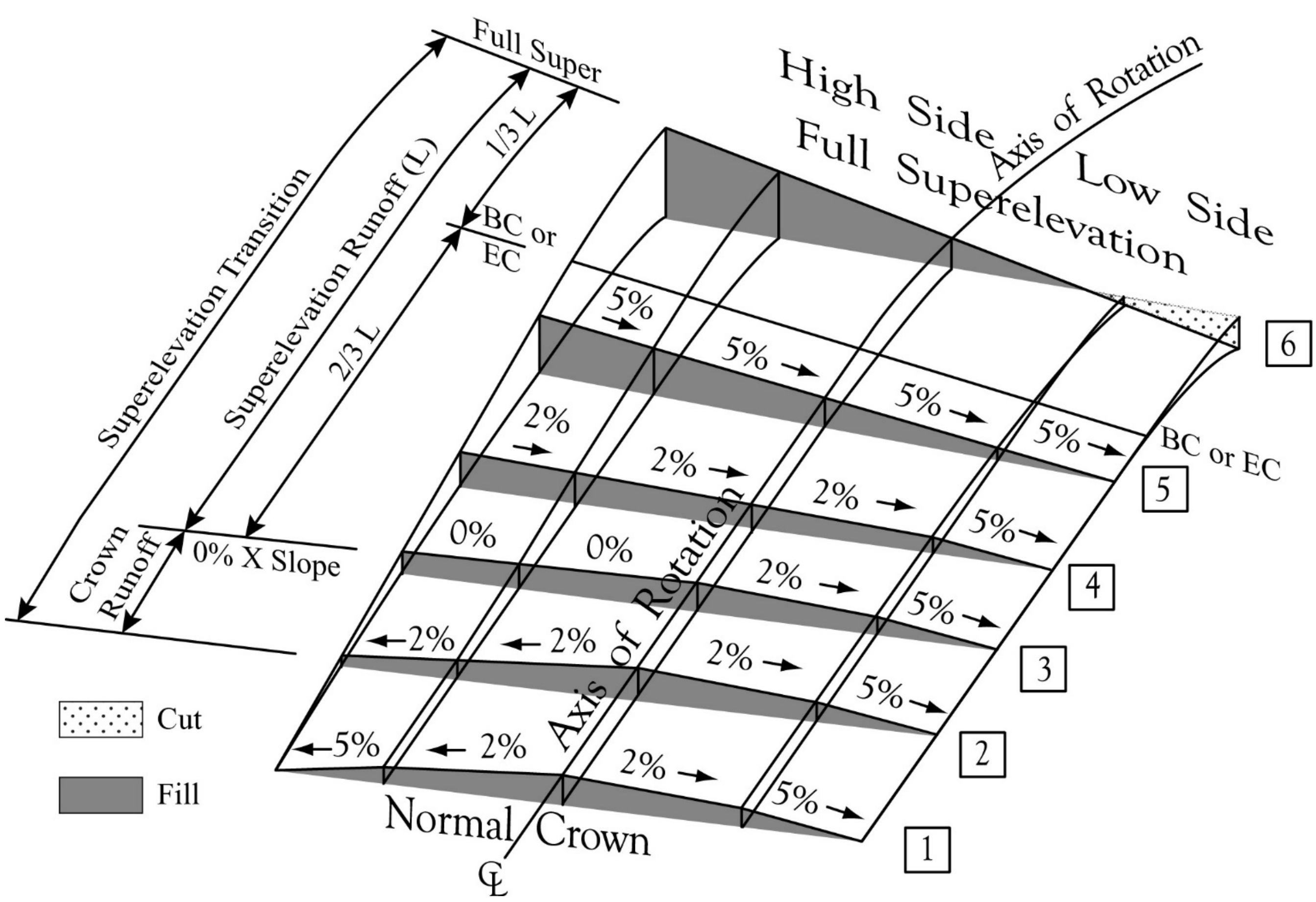
Superelevation Runoff Lengths

Superelevation Rate "e" ft/ft	2-Lane Highways & Multilane Ramps	Single Lane Ramps	Length, L (feet) Multilane Highways and Branch Connections with Various "D" Widths 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%.

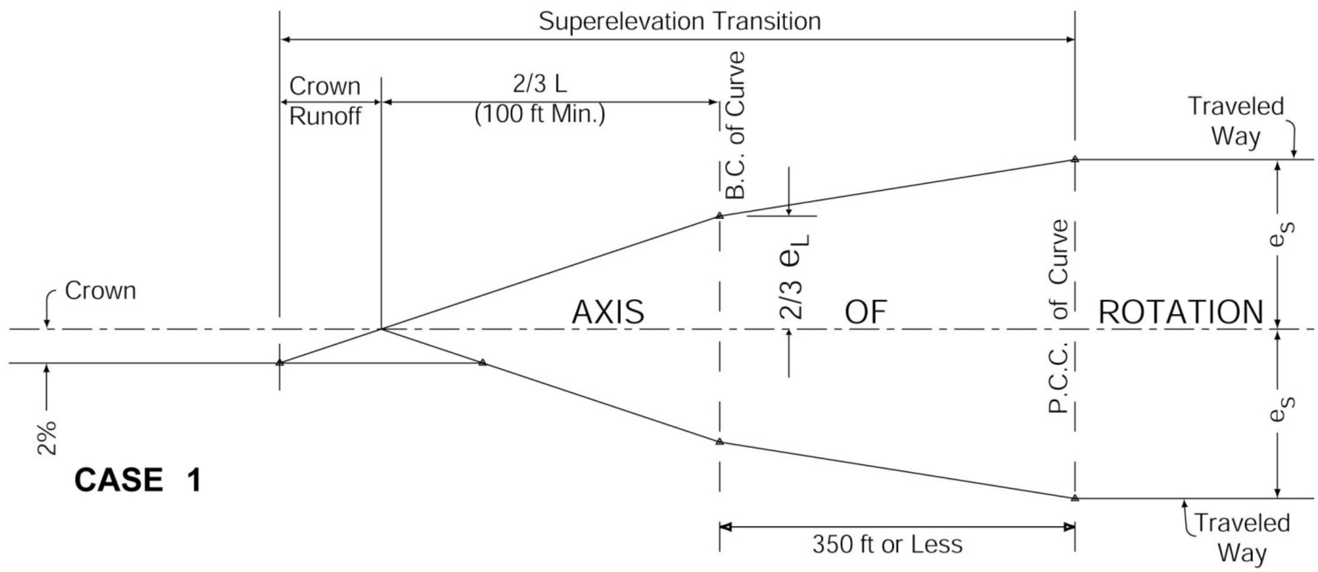


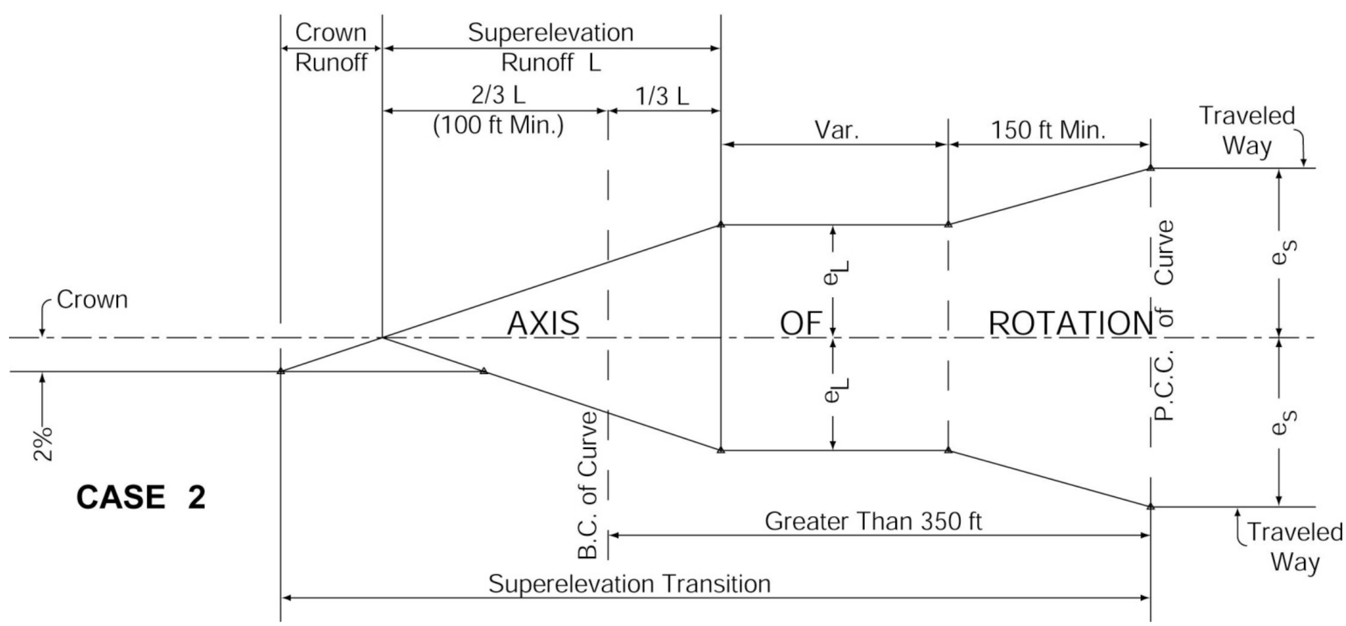
Elements of a Superelevation Transition (Right Curve)

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. <u>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.</u>

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. <u>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</u>

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 grade breaks within 200 feet should not exceed 0.5 percent.

Since flat vertical curves may develop poor drainage at the level section, adjusting the gutter grade or shortening the vertical curve may overcome any drainage problems.

On 2-lane roads, extremely long crest vertical curves, over one-half mile, should be avoided, since many drivers refuse to pass on such curves despite adequate sight distance. It is sometimes more economical to construct passing lanes than to obtain passing sight distance by the use of a long vertical curve.

Broken-back vertical curves consist of two vertical curves in the same direction separated by a short grade tangent. A profile with such curvature normally should be avoided, particularly in sags where the view of both curves is not pleasing.

204.5 Sustained Grades

(1) *General.* Maximum grade is not a complete design control. The length of an uphill grade is important as well, because it affects capacity, level of service, and delay when slow moving trucks, buses, and recreational vehicles are present.

A common criterion for all types of highways is to consider the addition of a climbing lane where the running speed of trucks falls 10 miles per hour or more below the running speed of remaining traffic. Figure 204.5 shows the speed reduction curves for a 200 lb/hp truck, which is representative of large trucks operating near maximum gross weight. The 10 miles per hour reduction criterion may be used as one method of determining need, however the Highway Capacity Manual should be consulted for detailed analysis.

(2) *Freeway Climbing Lanes.* If design year traffic volumes are expected to be near capacity, right of way acquisition and grading for a future lane should be considered at locations where the upgrade exceeds 2 percent and the total rise exceeds 50 feet.

Regardless of traffic volumes, the need for a climbing lane should be investigated on sustained upgrades greater than 2 percent if the total rise is greater than 250 feet. Refer to the Highway Capacity Manual for passenger car equivalent factors and sample calculations.

<u>Decision sight distance (Table 201.7) should be provided at climbing lane drops on freeways.</u>

(3) *Two-lane Road Climbing and Passing Lanes.* Climbing and passing lanes are most effective on uphill grades and curving alignment where the speed differential among vehicles is significant. Climbing and passing lanes should normally not be constructed on tangent sections where the length of tangent equals or exceeds the passing sight distance, because passing will occur at such locations without a passing lane and the double barrier stripe increases delay for opposing traffic. Where the ADT exceeds 5000, 4-lane passing sections may be considered. See Index 305.1(2) for median width standards.

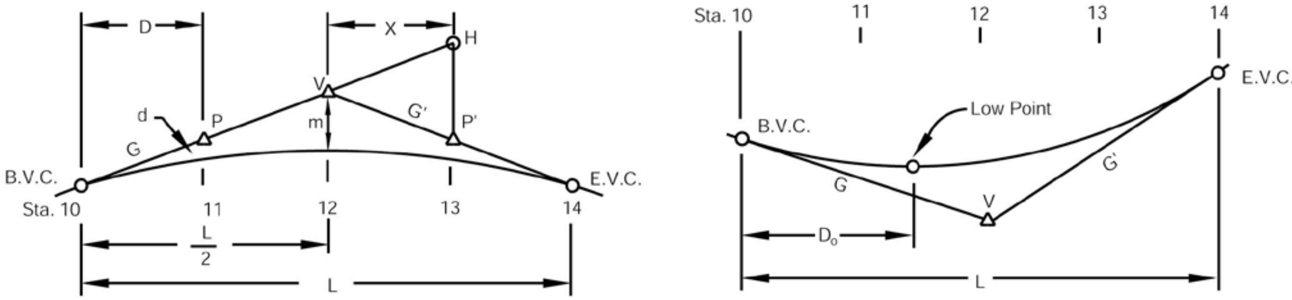
The Headquarters Division of Traffic Operations should be consulted regarding the length of climbing and passing lanes, which will vary with the design speed of the highway, the traffic volume, and other factors.

(4) *Turnouts*

(a) *General*. On a two-lane highway where passing is limited, the California Vehicle Code requires slow-moving vehicles followed by five or more vehicles to turn off at designated turnouts or wherever sufficient area for a safe turnout exists. Designated turnouts may be constructed in hilly or mountainous terrain or on winding roads in other areas.

Where less than 4-foot shoulders are provided on ascending grades, consideration should be given to providing several short sections of 4 feet or wider shoulder as turnouts for bicycle passing. Frequent turnouts that are at least 30 feet in length are recommended on sustained uphill grades. These turnouts will allow safe passing of bicycles by other bicyclists and vehicles in addition to providing

Figure 204.4
Vertical Curves



IN ANY VERTICAL CURVE :

(1) $$m = \frac{(G'-G)L}{8}$$

(2) $$m = \frac{1}{2}\left(\frac{El.B.V.C.+El.E.V.C.}{2} - El.V\right)$$

(3) $$d = m\left(\frac{D}{L/2}\right)^2 = \frac{4m}{L^2}D^2$$

(4) $$d = \frac{D^2(G'-G)}{2L} = \frac{-50D^2}{K}$$

(5) $$X = \frac{100(H-P')}{(G'-G)}$$

(6) $$S = G - D\left(\frac{G-G'}{L}\right) = G - \frac{100D}{K}$$

(7) $$D_O = \frac{LG}{G-G'}$$

(8) $$A = G - G'$$

(9) $$K = \frac{L}{A}(100) = \frac{L}{G-G'}(100)$$

WHERE:

L = Length of curve - measured horizontally - 100 ft. units or stations

G and G' = Grade rates - percent.

m = Middle ordinate - feet.

d = Correction from grade line to curve - feet.

D = Distance from B.V.C. or E.V.C. to any point on curve - stations.

S = Slope of the tangent to the curve at any point - percent.

X = Distance, from P' to V - feet.

H = Elevation of grade G projected to station of P'

P and P' = Elevation on respective grades.

D_o = Distance to low or high point from extremity of curve - stations.

K = Distance in feet required to achieve a 1% change in grade.

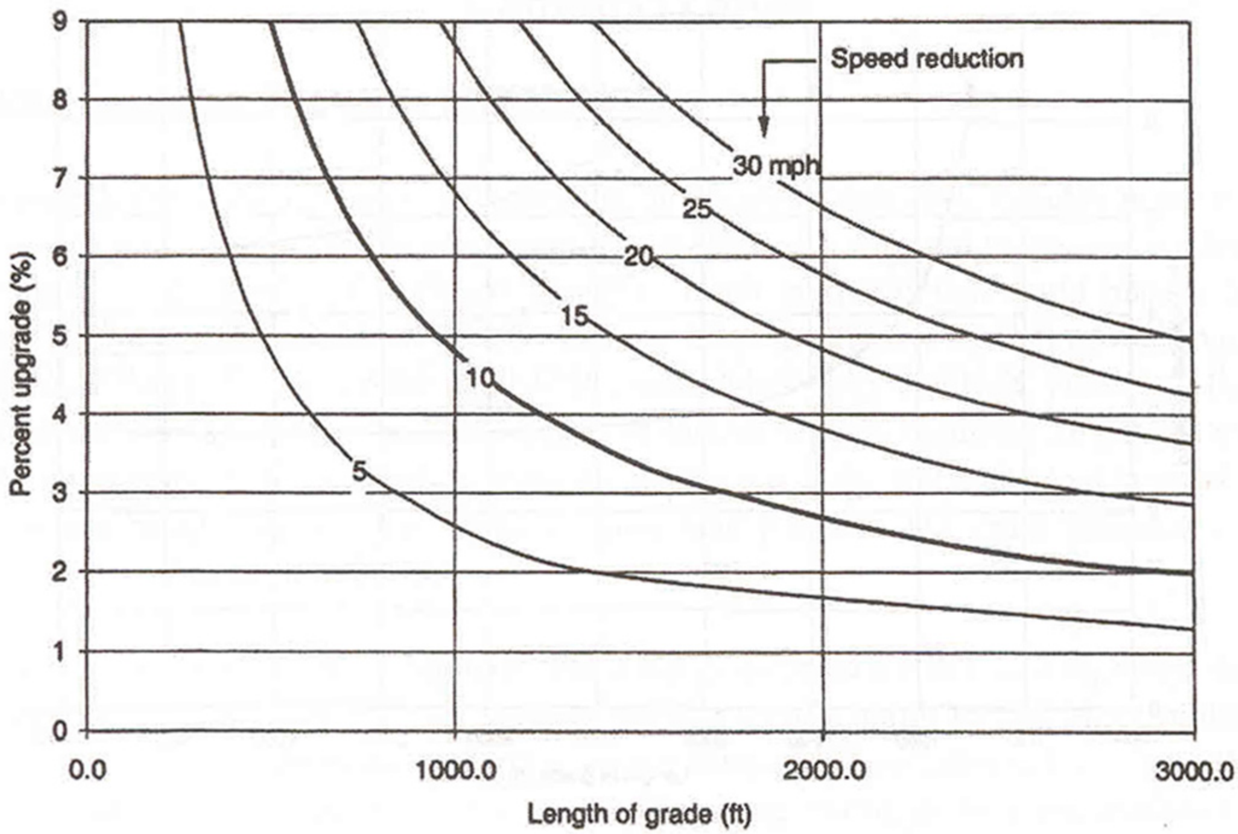
NOTES:

A rising grade carries a plus sign, while a falling grade carries a minus sign.

Thus, in a crest vertical curve as above, G carries a plus sign and G' carries a minus sign when progressing in the direction of the stationing. When progessing in the opposite direction, G becomes a minus grade and G' a plus grade.

Figure 204.5

Critical Lengths of Grade for Design



ASSUMED TYPICAL HEAVY TRUCK OF 200 lb/hp

resting opportunities on the sustained grade for bicyclists.

(b) *Length*. Designated turnouts should be from 200 feet to 500 feet long including a short taper (usually 50 feet) at each end. Approach speeds, grades, traffic volumes, and available space are some factors to be considered in determining the length. The Headquarters Traffic Liaison should be consulted if longer turnouts are desired.

(c) *Width*. Paved widths of at least 15 feet in fill sections and 12 feet in cut sections are recommended. Width is measured from the edge of traveled way. On the outside of curves along steep fill slopes or dropoffs, greater width or the installation of guardrail should be considered.

(d) *Location*. Turnouts should be located where there is stopping sight distance for approaching drivers to see vehicles leaving and re-entering the through lanes.

204.6 Coordination of Horizontal and Vertical Alignment

A proper balance between curvature and grades should be sought. When possible, vertical curves should be superimposed on horizontal curves. This reduces the number of sight restrictions on the project, makes changes in profile less apparent, particularly in rolling country, and results in a pleasing appearance. Where the change in horizontal alignment at a grade summit is moderate, a pleasing appearance may be attained by making the vertical curve overlap the horizontal curve.

When horizontal and vertical curves are superimposed, the combination of superelevation and profile grades may cause distortion in the outer pavement edges which could create drainage concerns or confuse drivers at night. In such situations edge of pavement profiles should be plotted and smooth curves introduced to eliminate any irregularities or distortion.

On highways in mountainous or rolling terrain where horizontal and vertical curves are superimposed at a grade summit or sag, the design speed of the horizontal curve should be at least equal to that of the crest or sag, and not more than 10 miles per hour less than the measured or estimated running (85th percentile) speed of vehicles on the approach roadway.

On long open curves, a uniform grade line should be used because a rolling profile makes for a poor appearance.

Horizontal and vertical curvature at intersections should be as flat as physical conditions permit.

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 a alignment consistency.

204.7 Separate Grade Lines

Separate or independent grade lines are appropriate in some cases for freeways and expressways.

They are not normally considered appropriate where medians are less than 65 feet wide (see Index 305.6). Exceptions to this may be minor differences between opposing grade lines in special situations.

In addition, for either interim or ultimate expressways, any appreciable grade differential between roadbeds should be avoided in the vicinity of at-grade intersections. For traffic entering from the crossroad, confusion and wrong-way movements could result if the pavement of the far roadway is obscured because of excessive grade differential.

204.8 Grade Line of Structures

(1) *Structure Depth.* The depth to span ratio for each structure is dependent on many factors. Some of these are: span, type of construction, aesthetics, cost, falsework limitations, and vertical clearance limitations. For purposes of preliminary planning and design, the depth to span ratios listed below may be used in setting grade lines at grade separations.

(a) Railroad Underpass Structures.

- Single track, through girder type structures: use 5-foot depth from top of rail to structure soffit (bottom of girder).

- 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.

- Effect of increased vertical clearance on the grade of adjacent sections.
- Closing local streets to all traffic or trucks only during construction.
- Detours.
- Carrying local traffic through construction on subgrade.
- Temporary or permanent lowering of the existing facility.
- Cost of higher clearance versus cost of traffic control.
- Desires of local agency.

Worker safety should be considered when determining vertical falsework clearance. Requests for approval of temporary vertical clearances less than 15 feet should discuss the impact on worker safety.

Temporary horizontal clearances less than shown in Table 204.8 or temporary vertical clearances less than 15 feet should be noted in the PS&E Transmittal Report.

To establish the grade of a structure to be constructed with a falsework opening, allowance must be made for the depth of the falsework. The minimum depths required for various widths of traffic opening are shown in Table 204.8.

Where vertical clearances, either temporary or permanent are critical, the District and the DES – Structure Design should work closely during the early design stage when the preliminary grades, structure depths, and falsework depths can be adjusted without incurring major design changes.

Where the vertical falsework clearance is less than 15 feet, advance warning devices are to be specified or shown on the plans. Such devices may consist of flashing lights, overhead signs, over-height detectors, or a combination of these or other devices.

Warning signs on the cross road or in advance of the previous off-ramp may be required for overheight permit loads. Check with the Regional Permit Manager.

After establishing the opening requirements, a field review of the bridge site should be made by the District designer to ensure that existing facilities (drainage, other bridges, or roadways) will not conflict with the falsework.

The placement and removal of falsework requires special consideration. During these operations, traffic should either be stopped for short intervals or diverted away from the span where the placement or removal operations are being performed. The method of traffic handling during these operations is to be included in the Special Provisions.

Topic 205 - Road Connections and Driveways

205.1 Access Openings on Expressways

Access openings are used only on expressways. The term access opening applies to openings through the right of way line which serve abutting land ownerships whose remaining access rights have been acquired by the State.

(1) *Criteria for Location.* <u>Access openings should not be spaced closer than one-half mile to an adjacent public road intersection or to another private access opening that is wider than 30 feet.</u> When several access openings are closely spaced, a frontage road should be considered (see Index 104.3). <u>To discourage wrong-way movements, access openings should be located directly opposite, or at least 300 feet from a median opening.</u>

Sight distance equivalent to that required for public road intersections shall be provided (see Index 405.1).

(2) *Width.* The normal access opening width should be 30 feet. A greater width may result in large savings in right of way costs in some instances, but should be considered with caution because of the possibility that public use might develop. Conversion of a private opening into a public road connection requires the consent of the CTC, which cannot be committed in advance (see the Project Development Procedures Manual).

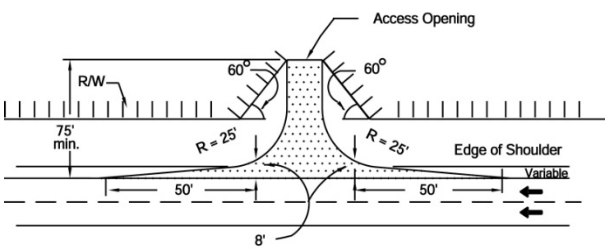
(3) *Recessed Access Openings.* Recessed access openings, as shown on Figure 205.1, are desirable at all points where private access is permitted and should be provided whenever they can be obtained without requiring alterations to existing adjacent improvements. When recessed openings are required, the opening should be located a minimum distance of 75 feet from the nearest edge of the traveled way.

(4) *Joint Openings.* A joint access opening serving two or more parcels of land is desirable whenever feasible. If the property line is not normal to the right of way line, care should be taken in designing the joint opening so that both owners are adequately served.

(5) *Surfacing.* All points of private access should be surfaced with adequate width and depth of pavement to serve the anticipated traffic. The surfacing should extend from the edge of the traveled way to the right of way line.

Figure 205.1

Access Openings on Expressways



RECESSED OPENING

NOTES:

- By widening the expressway shoulder, deceleration lanes may be provided where justified.
- This detail, without the recess, may be used on conventional highways.

205.2 Private Road Connections

The minimum private road connection design is shown on Figure 205.1. Sight distance requirements for the minimum private road connection are shown on Figure 405.7 (see Index 405.1(2)(c)).

205.3 Urban Driveways

These instructions apply to the design of driveways to serve property abutting on State highways in cities or where urban type development is encountered.

Details for driveway construction are shown on the Standard Plans. Corner sight distance requirements are not applied to urban driveways. See Index 405.1(2) for further information.

(1) *Correlation with Local Standards.* Where there is a local requirement regulating driveway construction, the higher standard will normally govern.

(2) *Driveway Width.* The width of driveways for both residential and commercial usage is measured at the throat, exclusive of any flares. ("W" as shown in Standard Plan A87A).

(3) *Residential Driveways.* The width of single residential driveways should be 12 feet minimum and 20 feet maximum. The width of a double residential driveway such as used for multiple dwellings should be 20 feet minimum and 30 feet maximum. The width selected should be based on an analysis of the anticipated volume, type and speed of traffic, location of buildings and garages, width of street, etc.

(4) *Commercial Driveways.* Commercial driveways should be limited to the following maximum widths:

   (a) When the driveway is used for one-way traffic, the maximum width should be 25 feet. If the driveway serves a large parcel, where large volumes of vehicles or large vehicles are expected, the entrance maximum width should be 40 feet and the exit maximum width should be 35 feet.

   (b) When the driveway is used for two-way traffic, the maximum width should be 35 feet. If the driveway serves a large parcel, where large volumes of vehicles or 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 Widenings

(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.

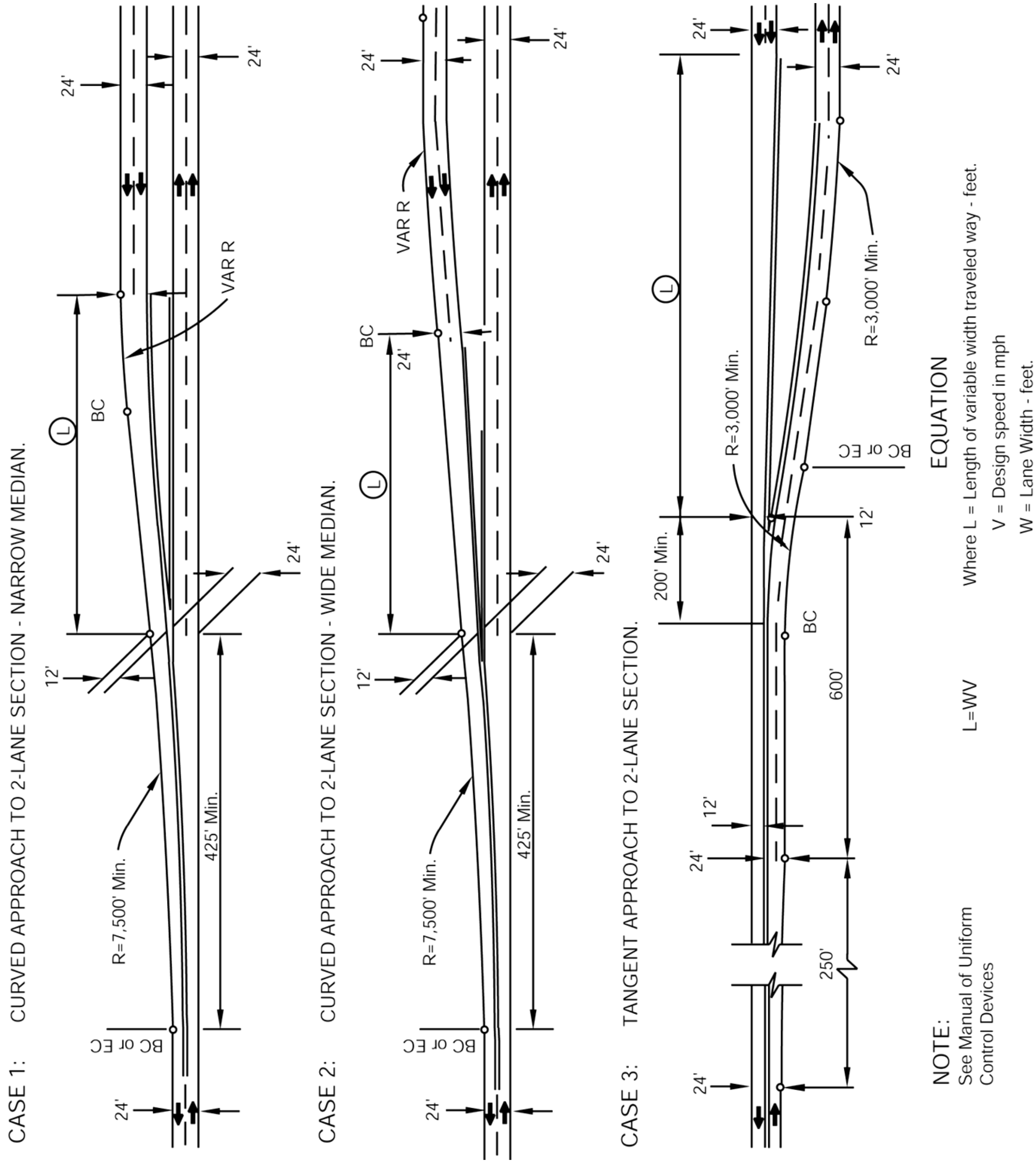
206.3 Pavement Reductions

(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



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 ¾WV. 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 District approval authority or Project Delivery Coordinator, depending upon the current District Design Delegation Agreement.

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.

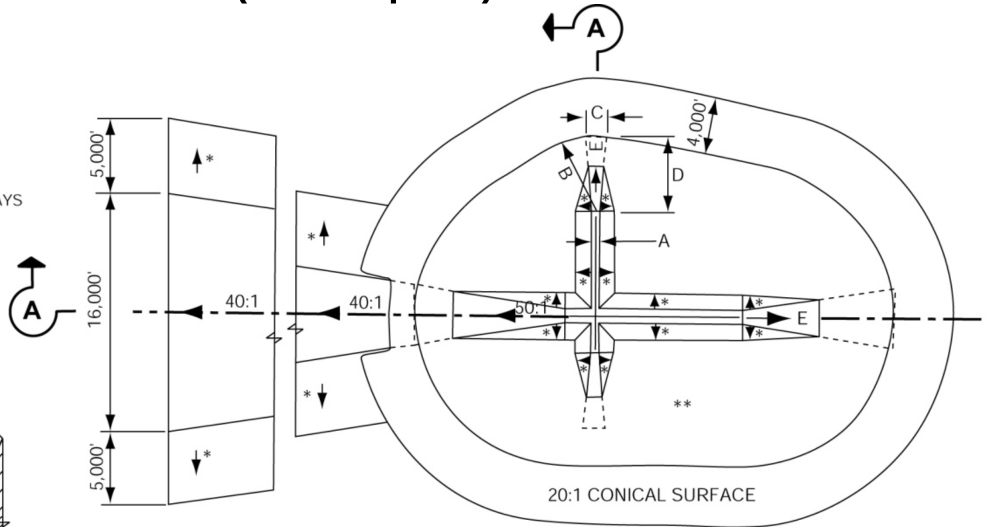
Notice to the FAA is required when highway construction is planned near an airport (civil or military) or a heliport. A "Notice of Proposed Construction or Alteration" should be submitted to the FAA Administrator when required under criteria listed in Paragraph 77.13 of the latest Federal Aviation Regulations, 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.

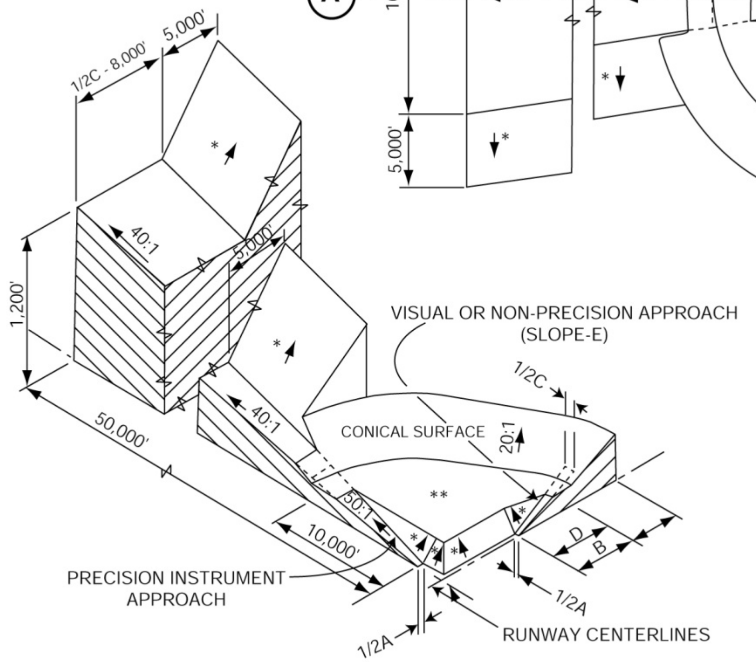
Figure 207.2A
Airway-Highway Clearance Requirements (Civil Airports)

* 7:1

** HORIZONTAL SURFACE 150' ABOVE ESTABLISHED AIRPORT ELEVATION

*** 16'-6" FREEWAYS
15'-0" CONVENTIONAL HIGHWAYS AND LOCAL ROADS
10'-0" PRIVATE ROADS





ISOMETRIC VIEW OF SECTION A-A

RUNWAY STANDARDS							
TYPES OF RUNWAY		VISUAL RUNWAY		NON-PRECISION INSTRUMENT RUNWAY			PRECISION INSTRUMENT RUNWAY
		I	II	I	II III	II IV	
DIM	ITEM	DIMENSIONAL STANDARDS (FEET)					
A	WIDTH OF PRIMARY SURFACE AND APPROACH SURFACE WIDTH AT INNER END	250	500	500	500	1,000	1,000
B	RADIUS OF HORIZONTAL SURFACE	5,000	5,000	5,000	10,000	10,000	10,000
C	APPROACH SURFACE WIDTH AT END	1,250	1,500	2,000	3,500	4,000	16,000
D	APPROACH SURFACE LENGTH	5,000	5,000	5,000	10,000	10,000	★
E	APPROACH SLOPE	20:1	20:1	20:1	34:1	34:1	★

I UTILITY RUNWAY
II RUNWAYS LARGER THAN UTILITY
III VISIBILITY MINIMUMS GREATER THAN 3/4 MILE
IV VISIBILITY MINIMUMS AS LOW AS 3/4 MILE
★ PRECISION INSTRUMENT APPROACH SLOPE IS 50:1 FOR INNER 10,000 FEET AND 40:1 FOR AN ADDITIONAL 40,000 FEET

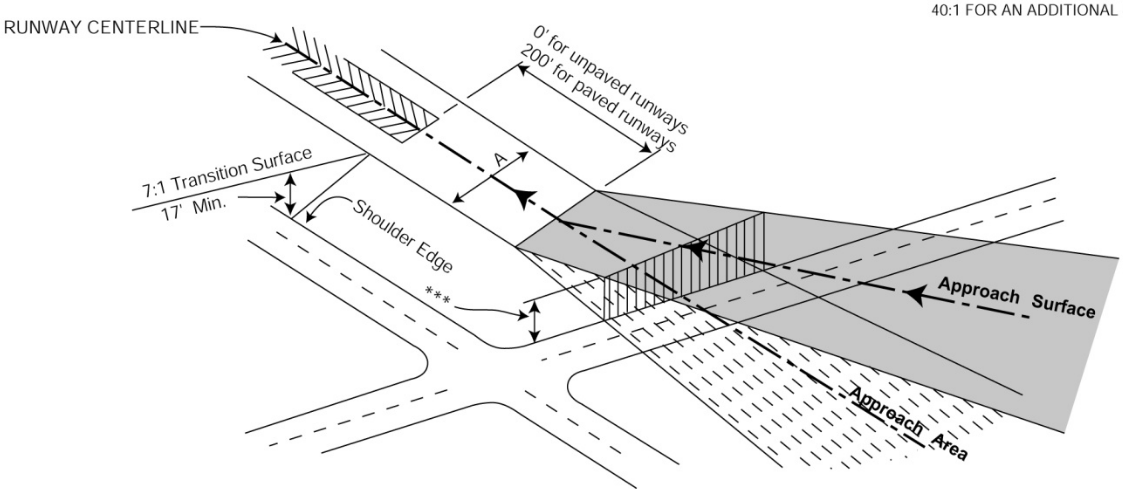
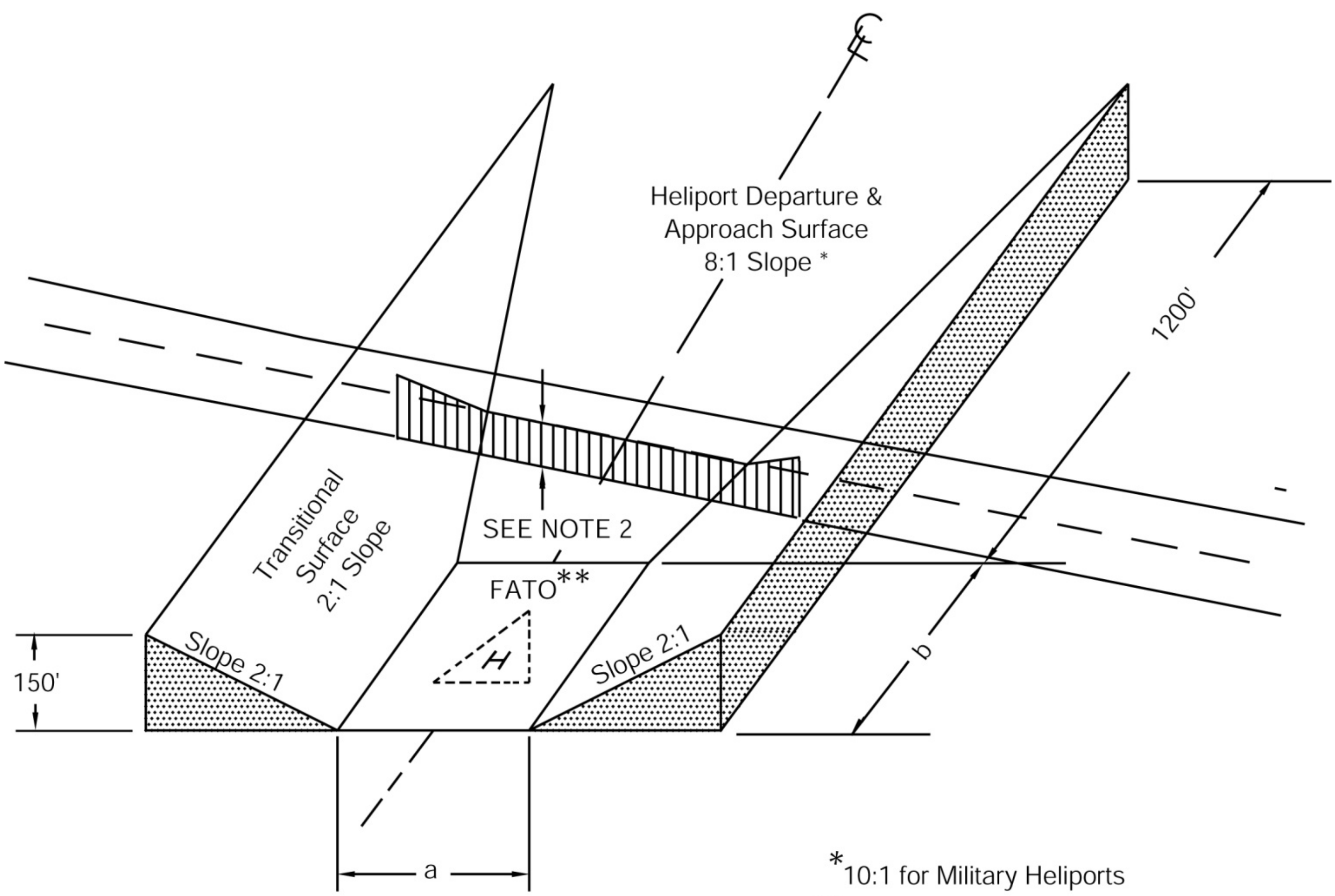


Figure 207.2B
Airway-Highway Clearance Requirements (Heliport)

NOTES:

1. FATO DIMENSIONS "a" AND "b" ARE EQUAL TO ONE-HALF TIMES THE OVERALL LENGTH OF THE DESIGN HELICOPTER, EXCEPT FOR TRANSPORT CATEGORY HELIPORTS, WHERE "a" EQUALS TWO TIMES THE ROTOR DIAMETER (100 FEET MIN.) AND "b" EQUALS TWO-TIMES THE ROTOR DIAMETER (200 FEET MIN.). CHECK WITH HELIPORT OWNER TO VERIFY HELICOPT CATEGORY.
2. MINIMUM VERTICAL CLEARANCE IS 16'-0" FOR FREEWAYS, 15'-0" FOR CONVENTIONAL HIGHWAYS AND LOCAL ROADS, AND 10'-0" FOR PRIVATE ROADS.
3. CONTACT THE HELIPORT OWNER/OPERATOR TO DETERMINE THE APPROVED APPROACH/DEPARTURE PATHS.

HIGHWAY CLEARANCE: PROFILE AT PAVEMENT EDGE NEAR AIRFIELD



*10:1 for Military Heliports

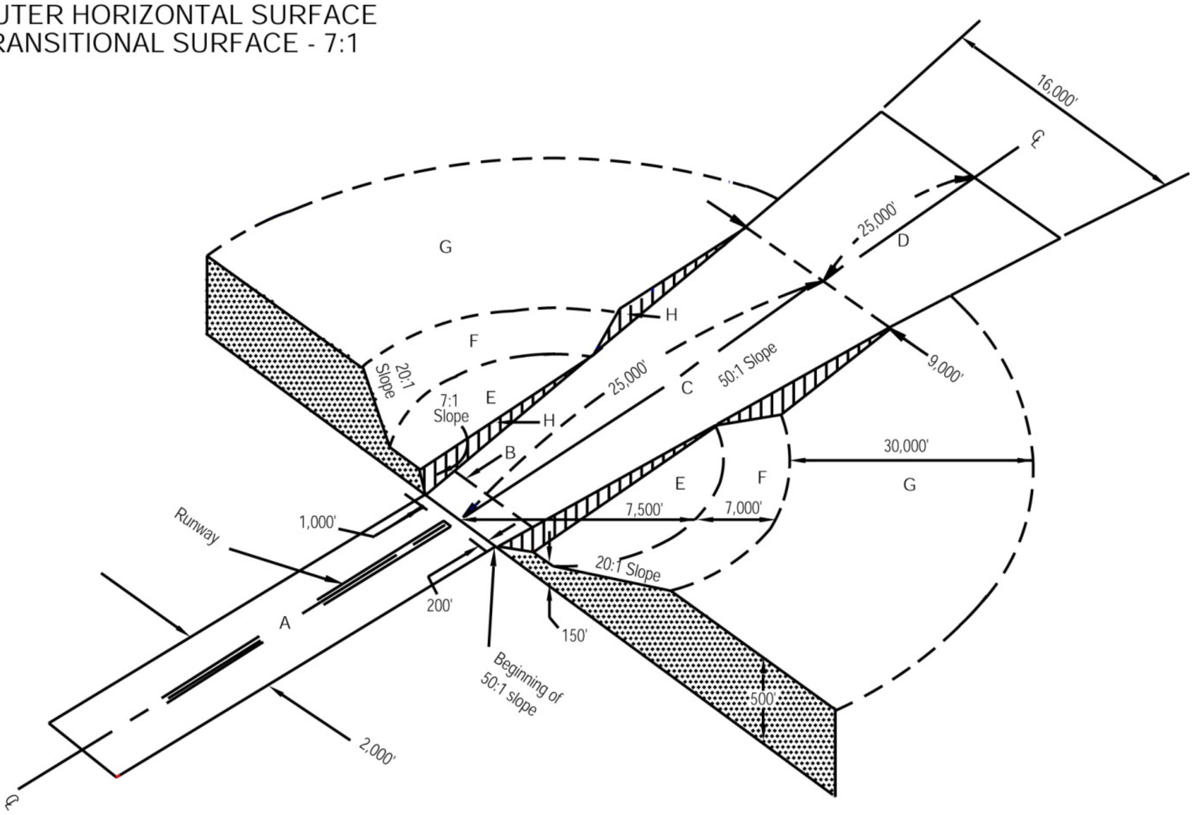
**Final Approach/Take Off Area

Figure 207.2C

Airway-Highway Clearance Requirements (Military Airports)

LEGEND

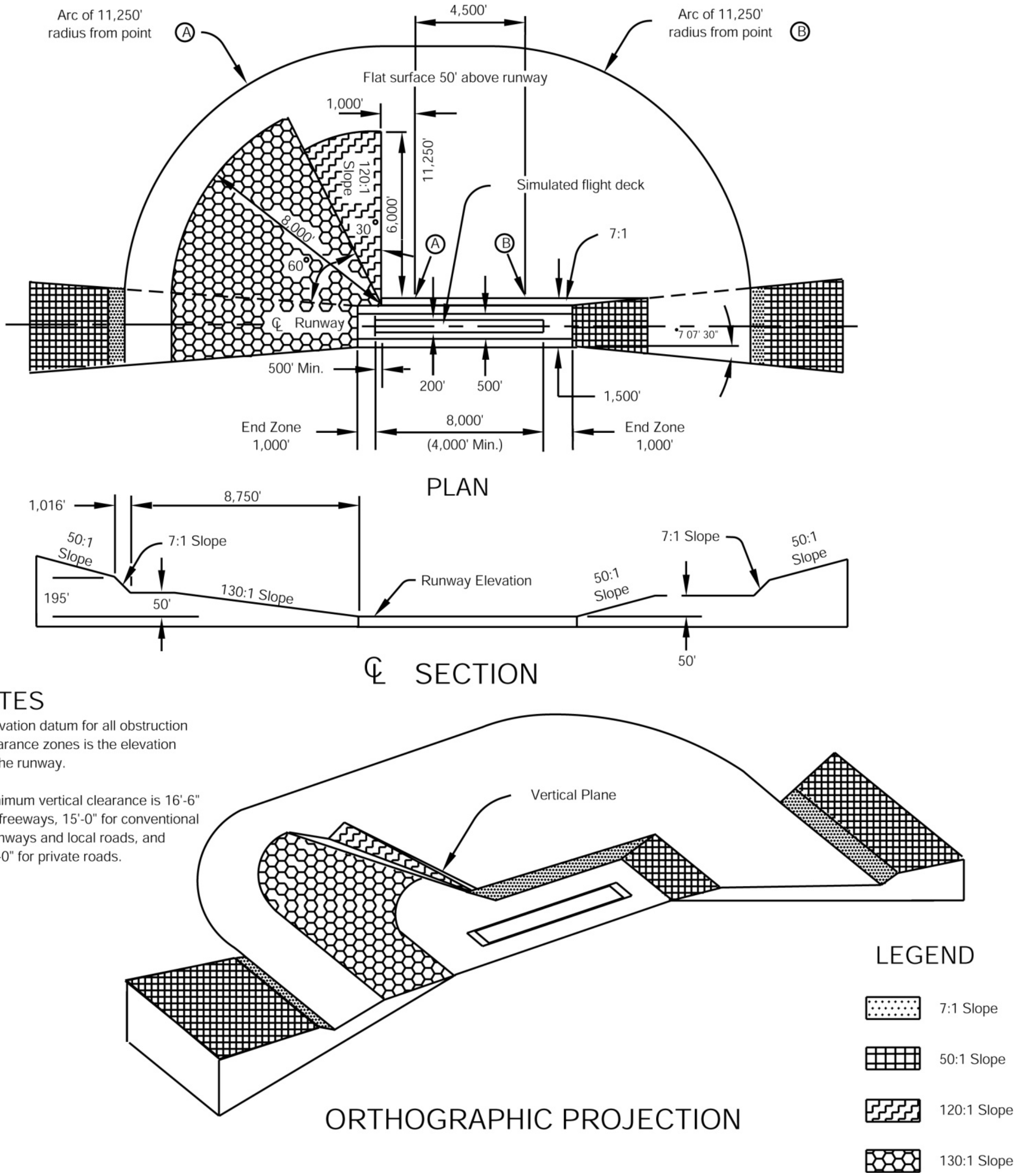
A- PRIMARY SURFACE
B- CLEAR ZONE SURFACE
C- APPROACH - DEPARTURE CLEARANCE SURFACE (GLIDE ANGLE) - 50:1
D- APPROACH - DEPARTURE CLEARANCE SURFACE (HORIZONTAL)
E- INNER HORIZONTAL SURFACE
F- CONICAL SURFACE - 20:1
G- OUTER HORIZONTAL SURFACE
H- TRANSITIONAL SURFACE - 7:1



NOTE:

MINIMUM VERTICAL CLEARANCE IS 16'-6" FOR FREEWAYS, 15'-0" FOR CONVENTIONAL HIGHWAYS AND LOCAL ROADS, AND 10'-0" FOR PRIVATE ROADS.

Figure 207.2D
Airway-Highway Clearance Requirements
(Navy Carrier Landing Practice Field)



NOTES
1. Elevation datum for all obstruction clearance zones is the elevation of the runway.

2. Minimum vertical clearance is 16'-6" for freeways, 15'-0" for conventional highways and local roads, and 10'-0" for private roads.

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

- Distance from project to nearest runway.
- Elevation of runway thresholds.
- Relationship between the proposed highway horizontal alignment and vertical profile to the nearest runway or heliport primary surface. Include elevations of objects referenced to the elevation of the end of the runway, such as overhead lights, signs, structures, landscaping, and vehicles.

One copy of FAA form 7460-1 should be forwarded to the Division of Design for information and one copy to the Division of Aeronautics for information and land use compatibility review.

Topic 208 – Bridges, Grade Separation Structures, and Structure Approach Embankment

208.1 Bridge Lane and Shoulder Width

(1) *State Highways.* **The clear width of all bridges, including grade separation structures, shall equal the full width of the traveled way and paved shoulders on the approaches with the following exceptions:**

(a) Bridges to be constructed as replacements on existing 2-lane, 2-way roads shall not have less than a 32-foot wide roadbed for ADT less than 400, and not less than 40-foot wide roadbed for ADT greater than 400. (see Index 307.2).

(b) When the approach shoulder width is less than 4 feet, the minimum offset on each side shall be 4 feet, and shall be documented in accordance with Index 82.2.

The width should be measured normal to the center line between faces of curb or railing measured at the gutter line. For offsets to safety shape barriers see Figure 208.1.

For horizontal and vertical clearances, see Topic 309.

(2) *Roads Under Other Jurisdictions.*

(a) Overcrossing Widths--(See Index 308.1.)

(b) Undercrossing Span Lengths--Initial construction should provide for the ultimate requirements. In areas where the local jurisdiction has a definite plan of development, the ultimate right of way width or at least that portion needed for the roadbed and sidewalks should be spanned.

If the undercrossing street or road has no median, one should be provided where necessary to accommodate left-turn lanes or the center piers of the undercrossing structure.

Where it appears that a 2-lane road will be adequate for the foreseeable future, but no right of way width has been established, a minimum span length sufficient for a 40-foot roadbed should be provided. Additional span length should be provided to permit future sidewalks where there is a foreseeable need. If it is reasonably foreseeable that more than two lanes will be required ultimately, a greater width should be spanned.

(c) For horizontal and vertical clearances, see Topic 309.

208.2 Cross Slope

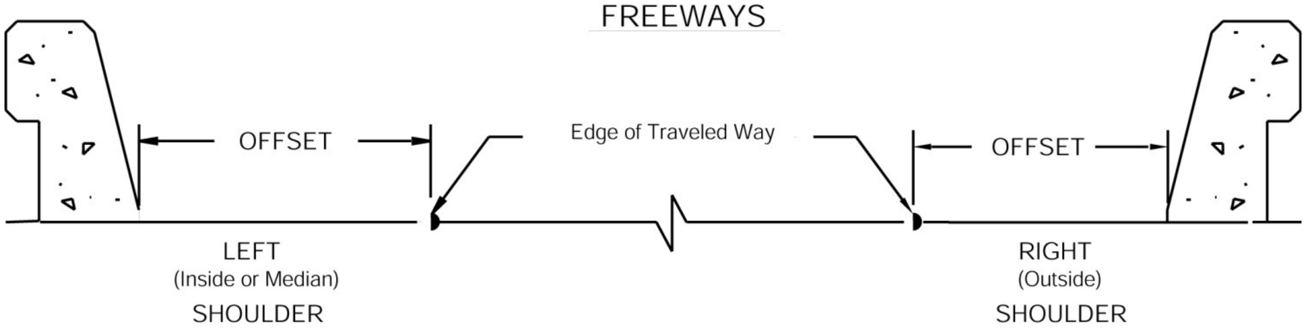
The crown is normally centered on the bridge except for one-way bridges where a straight cross slope in one direction should be used. The cross slope should be the same as for the approach pavement (see Index 301.2 and Index 203.9).

208.3 Median

<u>On multilane divided highways a bridge median that is 36 feet wide or less should be decked.</u> Exceptions require individual analysis. See Chapter 7 of the Traffic Manual for median barrier warrants.

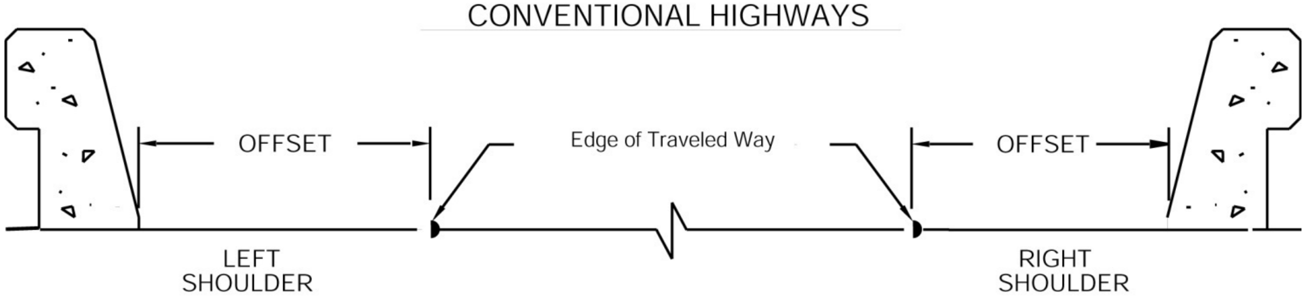
Figure 208.1
Offsets to Safety-Shape Barriers

FREEWAYS



Approach Shoulder Width	Left Shoulder	Right Shoulder
* 2' & 4' (Ramps)	4'	4'
5'	5'	5'
8'	8'	8'
10'	10'	10'

CONVENTIONAL HIGHWAYS



Approach Shoulder Width	Left Shoulder	Right Shoulder
* 2' & 4'	4'	4'
8'	8'	8'

* See Index 208.1(1)(b)

208.4 Bridge Sidewalks

Sidewalks on bridges should be provided wherever there are sidewalks or other pedestrian facilities that follow the highway. **The minimum width of a bridge sidewalk shall be 6 feet.** The recommended width should be 8 feet for pedestrian comfort. Bridges sidewalks in area types (see Index 81.2) with high levels of pedestrian activity may need to be greater than 8 feet (see Figure 208.10B).

208.5 Open End Structures

Embankment end slopes at open end structures should be no steeper than 1½:1 for all highways.

208.6 Bicycle and Pedestrian Overcrossings and Undercrossings

A bicycle overcrossing (BOC) or undercrossing (BUC) is a facility that provides a connection between bikeways or roads open to bicycling. They are considered Class I bikeways, or in certain situations may be considered Class IV bikeways. See Index 1003.1 for Class I bikeway guidance or DIB 89 for Class IV bikeways (separated bikeways) guidance.

A pedestrian overcrossing (POC) or undercrossing (PUC) is a facility that provides a connection between pedestrian walkways.

<u>The minimum width of walkway for pedestrian overcrossing should be 8 feet. The minimum vertical clearance of a pedestrian undercrossing should be 10 feet.</u> Skewed crossings should be avoided.

<u>Class I bikeways are designed for the exclusive use of bicyclists and pedestrians; equestrian access is prohibited.</u> See Chapter 1000 for Class I bikeway design guidance and Index 208.7 for equestrian undercrossing guidance. For additional information about the need to separate bicyclists from equestrian trails, see Index 1003.4.

POC's and PUC's must be designed to comply with DIB 82.

See Topic 309 for vertical clearances.

208.7 Equestrian Undercrossings and Overcrossings

Such structures should normally provide a clear opening 10 feet high and 10 feet wide. Skewed crossings should be avoided. The structure should be straight so the entire length can be seen from each end. Sustained grades should be a maximum of 10 percent. Decomposed granite or similar material should be used for the trail surface. While flexible pavement is permissible, a rigid pavement should not be used. See Index 1003.4 for separation between bicycle paths and equestrian trails. See DIB 82 for when trails are open to pedestrians.

Design guidance for equestrian overcrossings is pending.

208.8 Cattle Passes, Equipment, and Deer Crossings

Private cattle passes and equipment crossings may be constructed when economically justified by a right of way appraisal, as outlined in Section 7.09.09.00 of the Right of Way Manual.

The standard cattle pass should consist of either a standard box culvert with an opening 8 feet wide and 8 feet high or a metal pipe 120 inches in diameter. The invert of metal pipe should be paved with concrete or bituminous paving material.

If equestrian traffic is expected to use the culvert a minimum 10 feet wide by 10 feet high structure may be provided. However, the user of the facility should be contacted to determine the specific requirements.

If conditions indicate a reasonable need for a larger than standard cattle pass, it may be provided if economically justified by the right of way appraisal.

In some cases the installation of equipment or deer crossings is justified on the basis of public interest or need rather than economics. Examples are:

(a) A deer crossing or other structure for environmental protection purposes.

(b) Equipment crossings for the Forest Service or other governmental agencies or as a right of way obligation.

These facilities should be installed where necessary as determined by consultation with the appropriate affected entities.

A clear line of sight should be provided through the structure.

208.9 Railroad Underpasses and Overheads

Generally, it is desirable to construct overheads rather than underpasses whenever it is necessary for a highway and railroad to cross. Railroads should be carried over highways only when there is no other reasonable alternative.

Some undesirable features of underpasses are:

(a) They create bottlenecks for railroad operations.

(b) It is difficult to widen the highway.

(c) Pumping plants are often required to drain the highway.

(d) They are likely to lead to cost participation controversies for initial and future construction.

(e) Shooflies (temporary tracks) are generally required during construction.

(f) Railroads are concerned about the structure maintenance and liability costs they incur.

Advantages of overheads are:

(a) Railroads can use most of their right of way for maintenance.

(b) Overheads can be widened at a relatively low cost and with little difficulty.

(c) Less damage may be incurred in the event of a derailment.

(d) Agreements for design and maintenance can be reached more easily with railroads.

(e) Initial costs are generally lower.

The State, the railroads, and the public in general can usually benefit from the construction of an overhead structure rather than an underpass.

See Topic 309 for vertical clearances.

208.10 Bridge Barriers and Railings

(1) General. There are four classes of railings, each intended to perform a different function.

(a) Vehicular Barrier Railings--The primary function of these railings is to retain and redirect errant vehicles.

(b) Combination Vehicular Barrier and Pedestrian Railings--These railings perform the dual function of retaining both vehicles and pedestrians on the bridge. They consist of two parts--A concrete parapet barrier, generally with a sidewalk, and metal handrailing or fence-type railing.

(c) Pedestrian Railings--These railings prevent pedestrians from accidentally falling from the structure and, in the case of fence-type railing, reduce the risk of objects being dropped on the roadway below. See DIB 82 for additional requirements.

(d) Bicycle Railings--These railings retain bicycles and riders on the structure. They may be specifically designed for bicycles, or may be a combination type consisting of a vehicular barrier surmounted by a fence or metal handrail.

(2) Policies. To reduce the risk of objects being dropped or thrown upon vehicles, protective screening in the form of fence-type railings should be installed along new overcrossing structure sidewalks in urban areas (Sec.92.6 California Streets and Highways Code). Screening should be considered for the opposite side of structures having one sidewalk. Screening should be installed at such other locations determined to be appropriate.

Railings and barriers with sidewalks should not be used on structures with posted speeds greater than 45 miles per hour without barrier separation. All structure railings with a sidewalk in the Standard Plans are approved for posted speeds up to 45 miles per hour. **Any use of railings and barriers with sidewalks on structures with posted speeds greater than 45 miles per hour shall have a barrier separation between the roadway and the sidewalk.** The barrier separation type and the bridge rail selection requires approval by the HQ Traffic Liaison.

The approved types of railings for use on bridge structures are listed below and illustrated in Figures 208.10A, B, and C. Railing types not listed are no longer in general use; however, they may be specified in those cases where it is desirable to match an existing condition.

The District should specify in the bridge site data submittal the rail type to be used after consideration has been given to the recommendations of the local agency (where applicable) and the DES-SD.

(3) *Vehicular Barriers.* See Figure 208.10A.

(a) Concrete Barrier Type 732 and 736--These vehicular barriers are for general use adjacent to traffic. Figure 208.1 illustrates the position of the barrier relative to the edge of traveled way.

(b) Concrete Barrier Type 80--Use of this barrier requires approval by the HQ Traffic Liaison. It is intended for use in lower speed scenic areas where more see-through area is desired than is provided by a solid concrete parapet.

(4) *Combination Railings.* See Figure 208.10B.

(a) Barrier Railing Type 26--This is the barrier railing for general use when sidewalks are provided on a bridge. It must be accompanied with a tubular handrailing or a fence-type railing. See Index 208.4 for minimum width, however, this width may be varied as circumstances require.

(b) Barrier Railing Type 80SW--Similar to the Type 80, modified with a raised sidewalk and tubular handrailing. Use of this barrier requires approval by the HQ Traffic Liaison. It is intended for use in lower speed scenic areas where more see-through area is desired than is provided by a solid concrete parapet. The minimum sidewalk width is 6 feet; however, this width may be varied as circumstances require.

(c) Chain Link Railing Type 7--This is the fence-type railing for general use with Type 26 or Type 80SW barrier railing with sidewalk to reduce the risk of objects being dropped on the roadway below. When a sidewalk is provided on one side of a bridge and Type 732 barrier railing on the other side, Type 7 railing may be placed on top of the Type 732 as additional protection from dropped objects. Consideration should be given to the effect of the Type 7 railing on sight distance at the bridge ends and view over the side of the bridge. Lighting fixtures may be provided with Type 7 railings.

(d) Chain Link Railing Type 6--This railing may be used in lieu of Type 7 when special architectural treatment is required. It should not be used on curved alignment because of fabrication difficulties.

(e) Tubular Handrailing--This railing is used with Type 26, and Type 80SW to increase the combined rail height for the safety of pedestrians. It should be used in lieu of Type 7 where object dropping will not be a problem or at the ends of bridges to increase sight distance if fence-type railing would restrict sight distance.

(5) *Pedestrian Railings.* See Figure 208.10C

(a) Chain Link Railing Type 3--This railing is used on pedestrian structures to reduce the risk of objects being dropped on the roadway below.

(b) Chain Link Railing Type 7 (Modified)--This railing is similar to Type 7 except that it is mounted on the structure at the sidewalk level.

(c) Chain Link Railing --This railing is not as high as Types 3 or 7 and therefore, its use is restricted to those locations where object dropping or throwing will not be a problem.

(d) Chain Link Railing (Modification)--Existing railing may be modified for screening under the protective screening policy. The DES-SD should be contacted for details.

(6) Bicycle Railing. The minimum height of bicycle rail in certain circumstances is 48 inches; however, in most situations 42 inches above the deck surface is appropriate. Contact DES, Office of Design and Technical Services for more information. Pedestrian railings and combination railings consisting of a concrete barrier surmounted by a fence or tubular railing are satisfactory for bicycles, if a minimum 42-inch height is met. Bicycles are not considered to operate on a sidewalk, except in special cases where signs specifically direct cyclists to use a bike path or the sidewalk.

<u>As a general policy, bicycle railings should be installed at the following locations:</u>

(a) <u>On a Class I bikeway, except that a lower rail may be used if a curbed sidewalk, not signed for bicycle use, separates the bikeway from the rail or a shoulder at least 8 feet wide exists on the other side of the rail</u>.

(b) <u>On the outside of a Class II or III bikeway, unless a curbed sidewalk, not signed for bicycle use, separates the bikeway from the rail.</u>

(c) In other locations where the designer deems it reasonable and appropriate.

(7) Bridge Approach Railings. **Approach railings shall be installed at the ends of bridge railings exposed to approach traffic.**

Refer to Chapter 7 of the Traffic Manual for placement and design criteria of guardrail.

208.11 Structure Approach Embankment

(1) General. Structure approach embankment is that portion of the fill material within approximately 150 feet longitudinally of the structure. Refer to Figure 208.11A for limits, the Standard Specifications, and Standard Special Provisions for more information.

Quality requirements for embankment material are normally specified only in the case of imported borrow. When select material or local borrow for use in structure abutment embankments is shown on the plans, the Resident Engineer (RE) is responsible for assuring the adequacy of the quantity and quality of the specified material. The Project Engineer should include adequate information and guidance in the RE File to assist the RE in fulfilling this responsibility.

(2) Foundations and Embankment Design. Overall performance of the highway approach to the bridge depends, to a significant degree, upon the long-term settlement/consolidation of the approach foundation and structure abutment embankment. A design that minimizes this post construction settlement/consolidation is essential. Factors that influence settlement/consolidation include soil types and depths, static and dynamic loads, ground water level, adjacent operations, and changes in any of the above. The PE must follow the foundation and embankment recommendations by the Division of Engineering Services, Geotechnical Services (DES-GS) and District Materials Engineer (DME). The DME and/or DES-GS must approve any deviations from their recommendations including Construction Change Orders (CCO's).

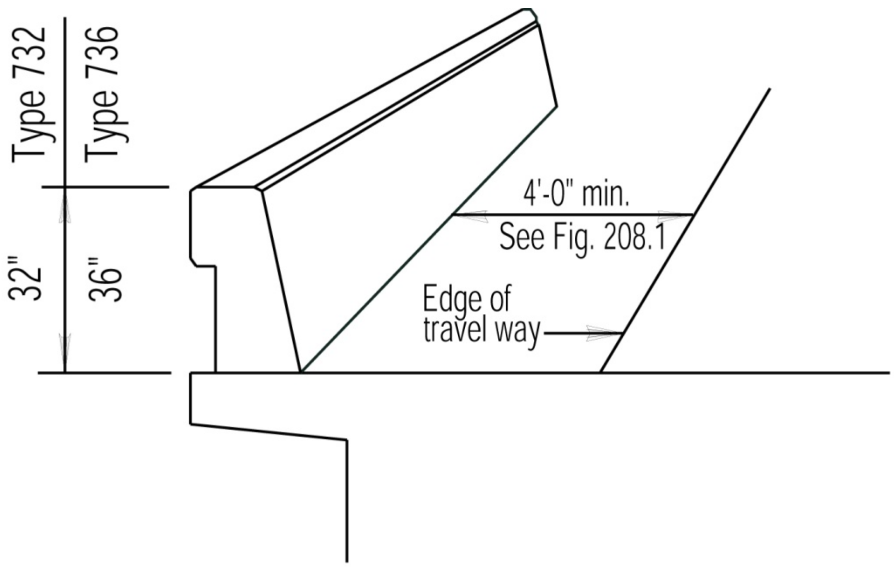
The relative compaction of material within the embankment limits must be at least 95 percent, except for the outer 5 feet of embankment measured horizontally from the side slope (see Figure 208.11A). The DME and/or OSF may recommend using select material, local and/or imported borrow to assure that the compaction requirements are met and that shrink/swell problems are avoided. They may also recommend a height and duration of embankment surcharge to accelerate foundation consolidation.

Poor quality material, such as expansive soils, must be precluded from structure abutment embankments unless treated. If sufficient quality roadway excavation material is unavailable for constructing of structure abutment embankments, the designer may specify select material, local borrow, or imported borrow to satisfy the design requirements.

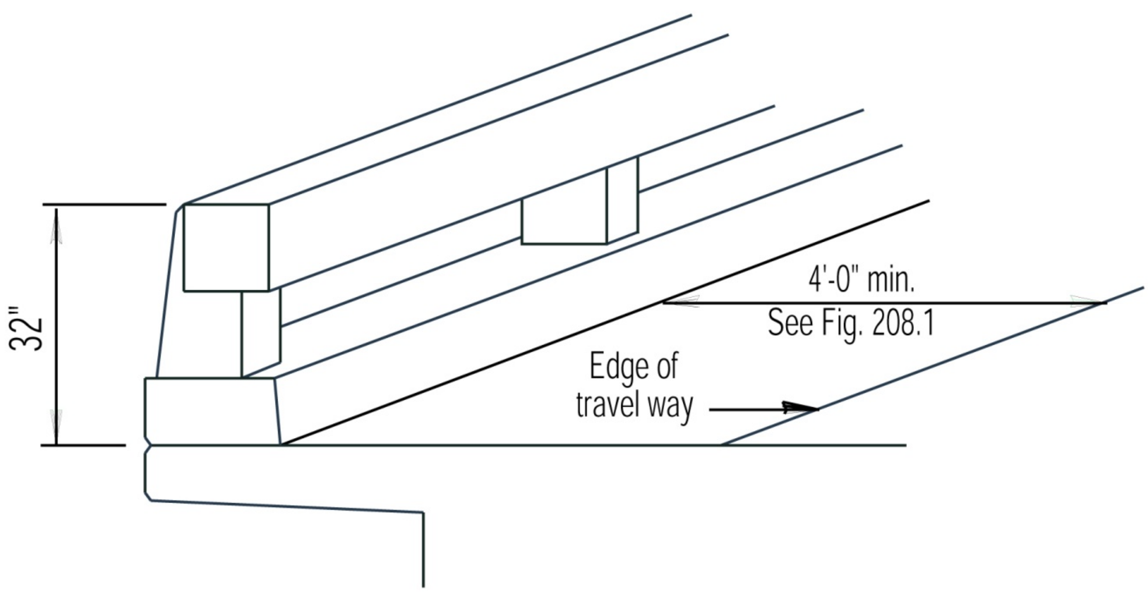
(3) Abutment Drainage. Special attention must be given to providing a positive drainage

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

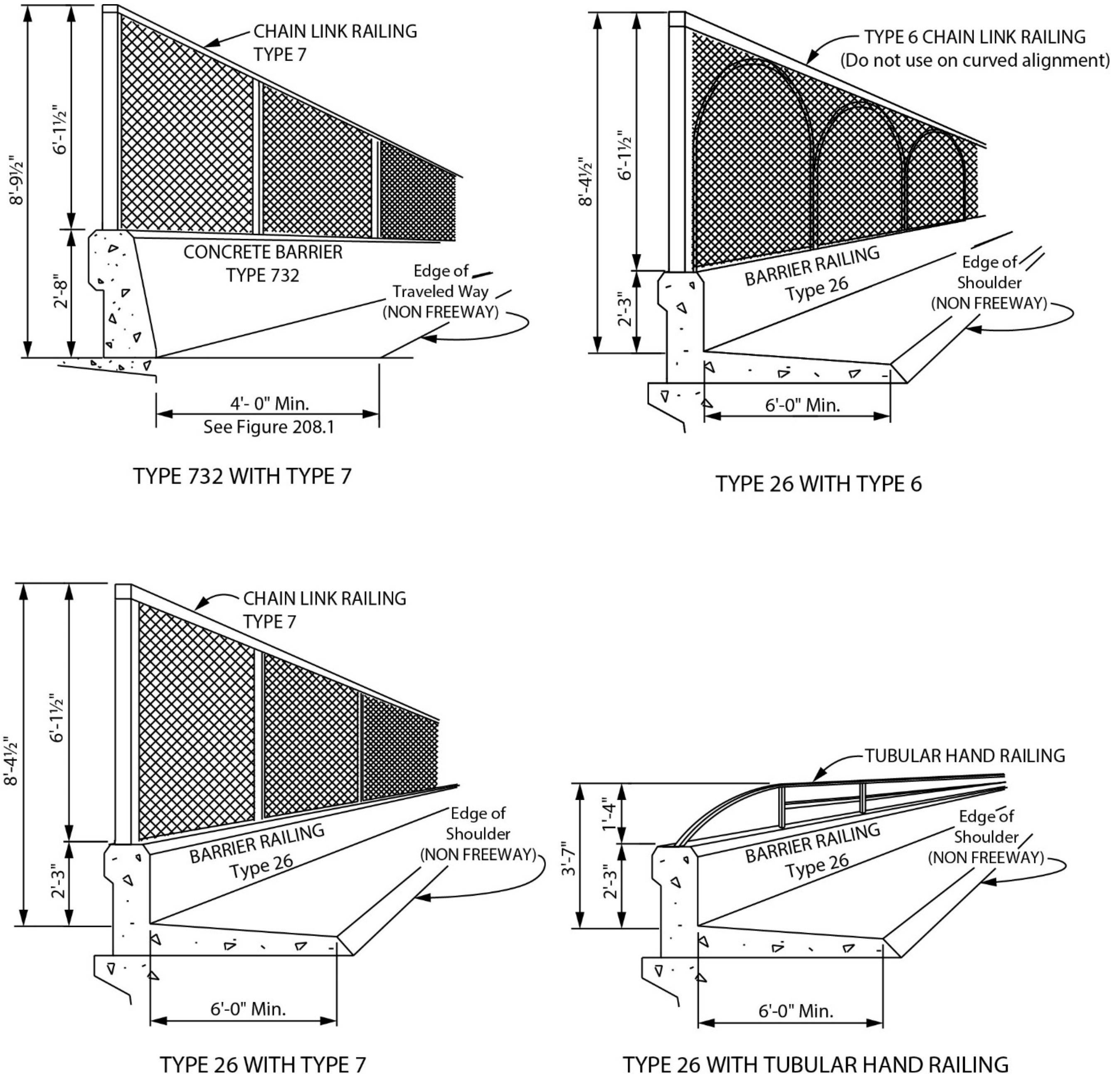
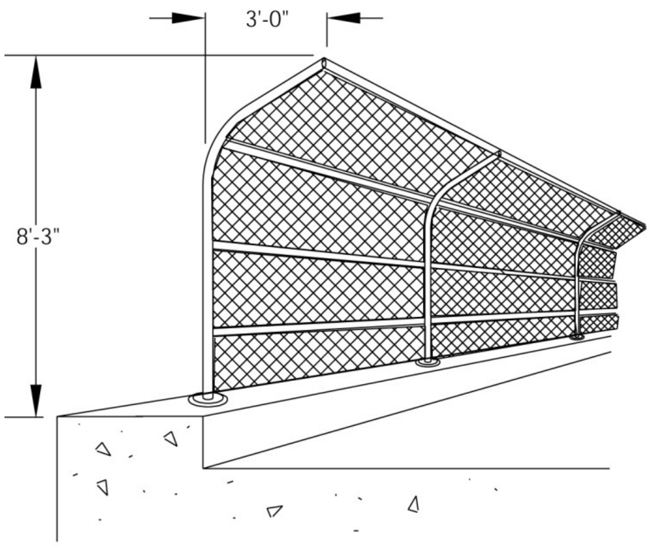
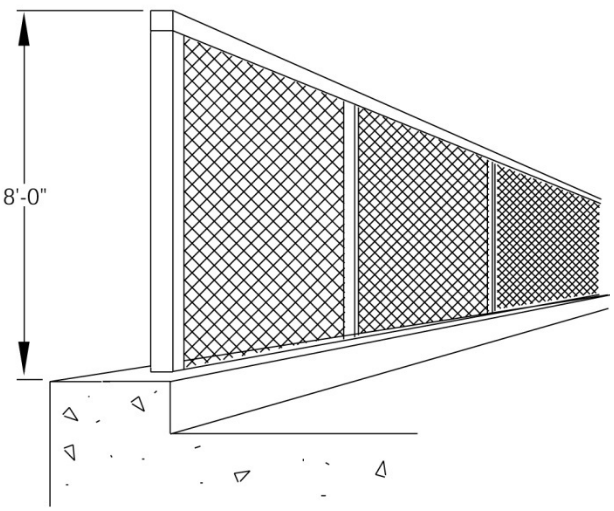


Figure 208.10C

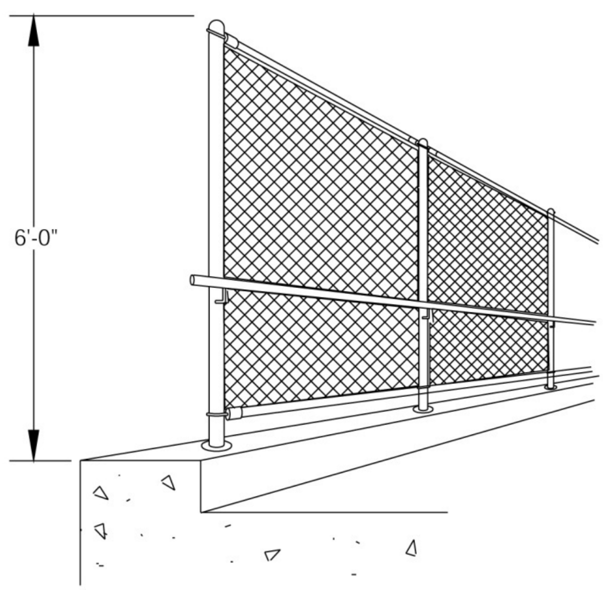
Pedestrian Railings for Bridge Structures



CHAIN LINK RAILING TYPE 3



CHAIN LINK RAILING TYPE 7 (MODIFIED)



CHAIN LINK RAILING

Figure 208.11A

Limits of Structure Approach Embankment Material

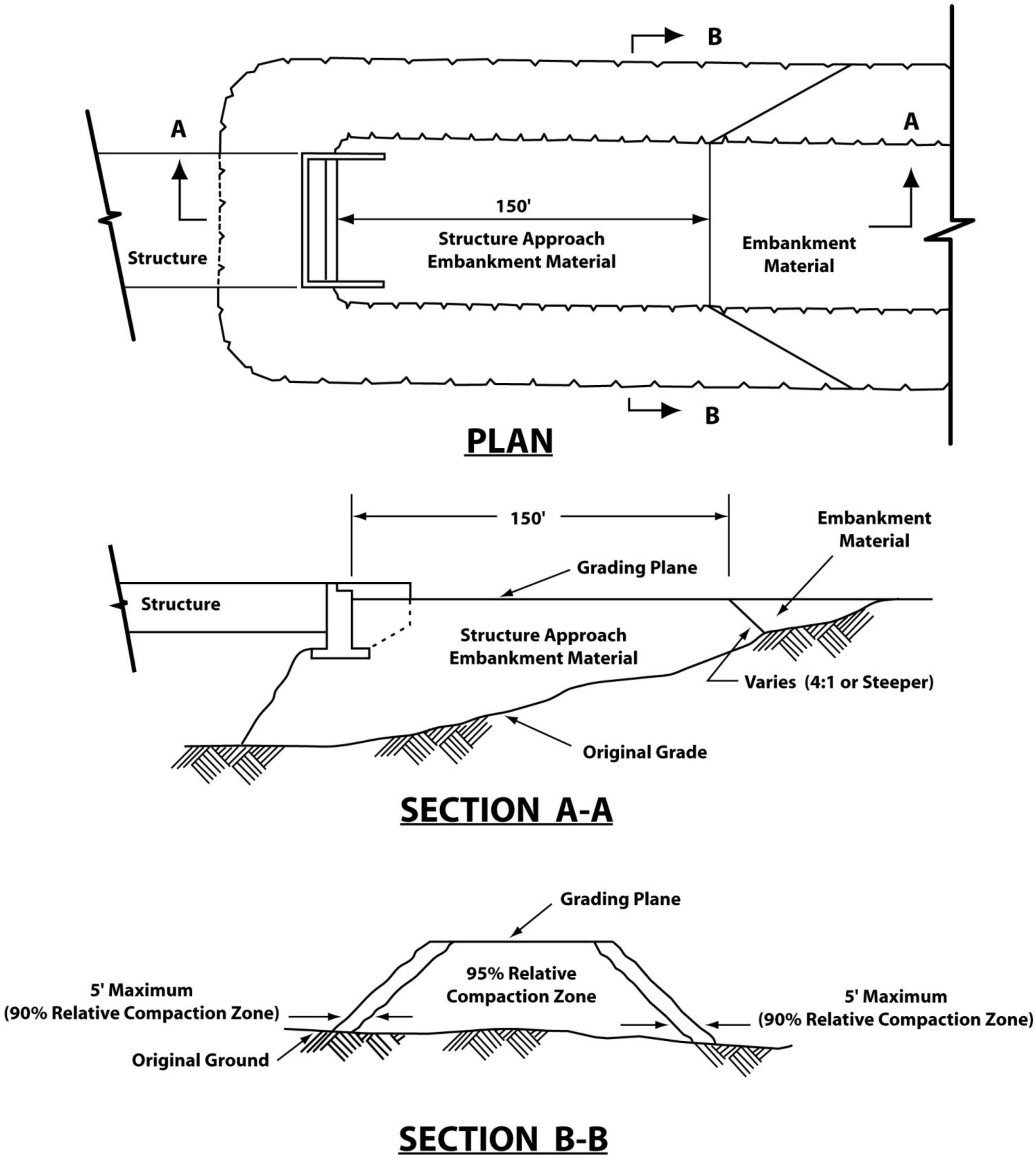
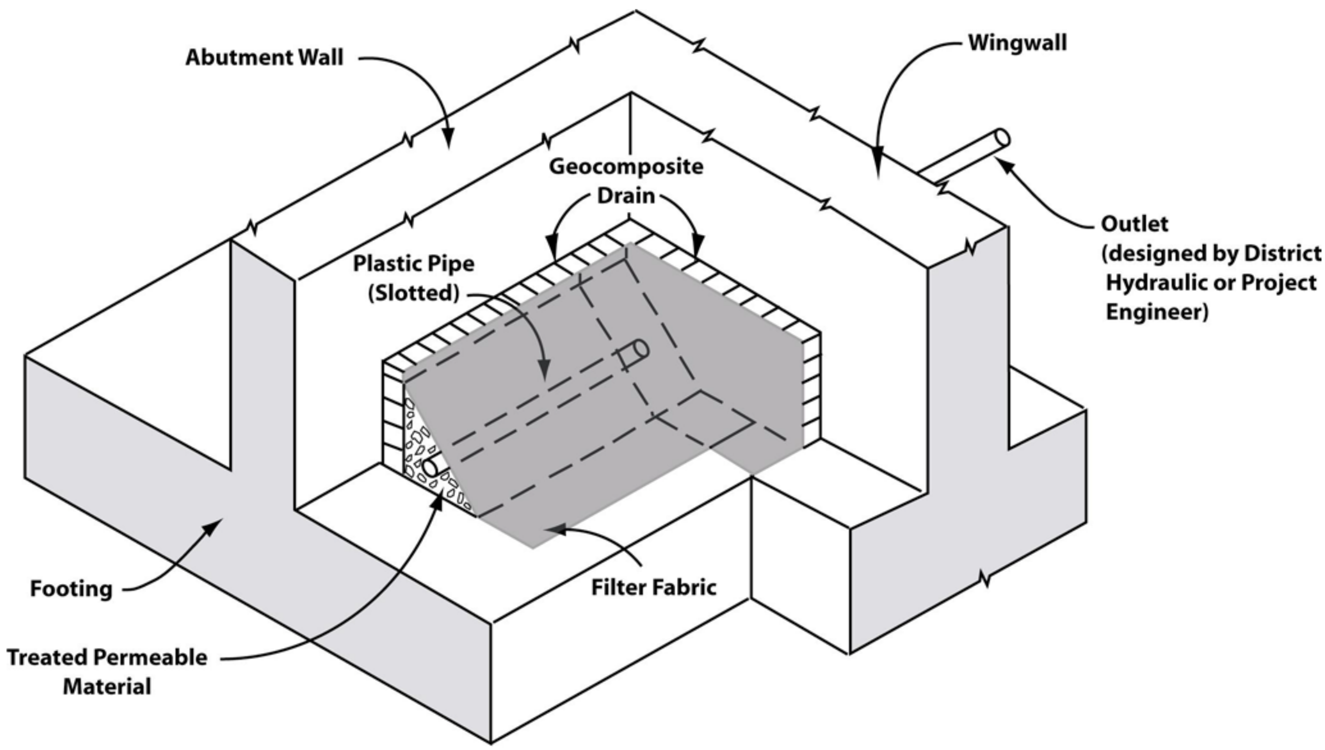


Figure 208.11B

Abutment Drainage Details



NOTES:

1. Applicable to new construction only.
2. Reference Structures Design Standard Detail XS22-17
3. All details shown are designed by the DES except where noted otherwise.
4. Outlet may be in wingwall of abutment wall.

system that minimizes the potential for water damage to the structure approach embankment, see Chapter 870 for further details. The Division of Engineering Services (DES), Structures Design (DES-SD) is responsible for the design of the structure approach drainage system, which includes:

- A geocomposite drain covered with filter fabric placed behind both the abutment wall and wingwalls, as indicated in Figure 208.11B.
- A slotted plastic pipe drain, encapsulated with treated permeable material, placed along the base of the inside face of the abutment wall as illustrated in Figure 208.11B.

(4) Slope Treatment. See Topic 707, Slope Treatment Under Structures, for guidance regarding the treatment of bridge approach end slopes.

The District Hydraulic Engineer or Project Engineer must design a pipe outlet that ties into the structure approach drainage system as it exits the structure. A pipe outlet system should carry the collected water to a location where it will not cause erosion. Storm Water Best Management Practices should be incorporated. For further information on Storm Water Management, visit the Division of Design Storm Water website.

Coordination with DES is necessary for the exit location of the pipe system. The outlet type should be chosen from the standard edge drain outlet types shown in the Standard Plans or tied into an underground drainage system. The PE must review the drainage design to ensure the adequacy of the drainage ties between the structure approach drainage system and either new or existing drainage facilities. For alternative details, see Bridge Design Aids.

Topic 209 - (currently not in use)

Topic 210 - Reinforced Earth Slopes and Earth Retaining Systems

210.1 Introduction

Constructing roadways on new alignments, widening roadways on an existing alignment, or repairing earth slopes damaged by landslides are situations that may require the use of reinforced earth slopes or earth retaining systems. Using cut and embankment slopes that are configured at slope ratios that are stable without using reinforcement is usually preferred; however, topography, environmental concerns, and right of way (R/W) limitations may require the need for reinforced earth slopes or an earth retaining system.

The need for reinforced earth slopes or an earth retaining system should be identified as early in the project development process as possible, preferably during the Project Initiation Document (PID) phase.

210.2 Construction Methods and Types

(1) Construction Methods.

Both reinforced earth slopes and earth retaining systems can be classified by the method in which they are constructed, either *top-down* or *bottom-up*.

- “Top-down” construction –This method of construction begins at the top of the reinforced slope or earth retaining system and proceeds in lifts to the bottom of the reinforced slope or earth retaining system.

  If required, reinforcement is inserted into the in situ material during excavation.

- “Bottom-up” construction – This method of construction begins at the bottom of the reinforced slope or earth retaining system, where a footing/leveling pad is constructed, construction then proceeds towards the top of the reinforced slope or earth retaining system. If required, reinforcement is placed behind the face of the reinforced slope or earth retaining system. It should be noted that if a “Retaining Wall” earth retaining system is to be used in a cut situation, a temporary back cut or shoring system is required behind the wall.

The District Project Engineer (PE) should conduct an initial site visit and assessment to determine all potential construction limitations. The preferred construction method is top-down due to the reduced shoring, excavation and backfilling. However, this method is not always available or appropriate based on the physical and geotechnical site conditions. The site should also be examined for R/W or utility constraints that would restrict the type of excavation or limit the use of some equipment. In addition, the accessibility to the site for construction and contractor staging areas should be considered.

Table 210.2 summarizes the various reinforced earth slopes and earth retaining systems that are currently available for use, along with the method in which they are constructed.

(2) Reinforced Earth Slopes (PS&E by District PE)

Reinforced earth slopes incorporate metallic or non-metallic reinforcement in construction of embankments and cut slopes with a slope angle flatter than 70 degrees from the horizontal plane. Reinforced earth slopes should be used in conjunction with erosion mitigation measures to minimize future maintenance costs. The slope face is typically erosion protected with the use of systems such as geosynthetics, bio-stabilization, rock slope protection, or reinforced concrete facing.

(3) Earth Retaining Systems

Earth retaining systems can be divided into five major categories depending upon the nature of the design and whether they are designed by the owner (State designed), a Proprietary vendor or a combination thereof. The term "State designed" as referenced herein is utilized to encompass earth retaining systems that are designed by the State or by Local or Private entities on behalf of the State. No assignment of roles and responsibilities is intended. The five categories are as follows:

(a) State Designed Earth Retaining Systems which utilize Standard Plans (PS&E by District PE).

Standard Plans are available for a variety of earth retaining systems (retaining walls). Loading conditions and foundation requirements are as shown on the Standard Plans. For sites with requirements that are not covered by the Standard Plans, a special design is required. To assure conformance with the specific Standard Plan conditions and requirements, and subsequent completion of the PS&E in a timely fashion, the District PE should request a foundation investigation for each location where a retaining wall is being considered. Retaining walls that utilize Standard Plans are as follows:

- Retaining Wall Types 1 and 1A (Concrete Cantilever). These walls have design heights up to 36 feet and 12 feet respectively, but are most economical below 20 feet. Concrete cantilever walls can accommodate traffic barriers, and drainage facilities efficiently. See Standard Plans for further details.

Retaining Wall Type 5 (Concrete L-Type Cantilever). This wall has a design height up to 12 feet. Although more costly than cantilever walls, these walls may be required where site restrictions do not allow for a footing projection beyond the face of the wall stem. See Standard Plans for further details.

Retaining Wall Type 6 (Concrete Masonry Walls). These walls may be used where the design height of the wall does not exceed 6 feet. These walls are generally less costly than all other standard design walls or gravity walls. Where traffic is adjacent to the top of the wall, guardrail should be set back as noted in the Standard Plans. See Standard Plans or further details.

- Crib Walls. The following types are available:

  Concrete Crib Wall - This type of crib wall may be used for design heights up to 50 feet. Concrete crib walls are suited to coastal areas and higher elevations where salt air and deicing salts may limit the service life of other types of crib walls. See Standard Plans or further details.

  Steel Crib Wall - This type of crib wall may be used for design heights up to 36 feet. Steel crib walls are light in weight; easily transported and installed; and, therefore, suited for relatively inaccessible installations and for emergency repairs. See Standard Plans for further details.

  Concrete crib walls constructed on horizontal alignments with curves or angle points require special details, particularly when the wall face is battered. Because crib wall faces can be climbed, they are not recommended for use in urban locations where they may be accessible to the public.

(b) State Designed Earth Retaining Systems which requires Special Designs.

Some locations will require a special design to accommodate ground contours, traffic, utilities, man-made features, site geology, economics, or aesthetics.

Some special design earth retaining systems are as follows:

- Standard Plan Walls (PS&E by Structure PE). The design loadings, heights, and types of walls in the Standard Plans cover frequent applications for earth retaining systems. However, special designs are necessary if the imposed loading exceeds that shown on the Standard Plan. Railroad live loads; building surcharge; loads imposed by sign structures, electroliers, or noise barriers are examples of loading conditions that will require special designs. Foundation conditions that require pile support for the wall and angle points in the wall geometry necessitate a special design.

- Non-Gravity Cantilevered Walls (PS&E by Structure PE). These walls include sheet pile walls, soldier pile walls with lagging, tangent soldier pile walls, secant soldier pile walls, slurry diaphragm walls, and deep soil mixing walls. These walls are most practical in cut sections and are best suited for situations where excavation for a retaining wall with a footing is impractical because of traffic, utilities, existing buildings, or R/W restrictions. In embankment sections, a non-gravity cantilevered wall is a practical solution for a roadway widening where design heights are less than 15 feet. They are also practical for slip-out corrections. Non-gravity cantilevered walls can consist of concrete, steel, timber, or cemented soil piles that may be either driven into place or placed in drilled holes and trenches.

- Anchored Walls (PS&E by Structure PE). These walls are typically composed of the same elements as non-gravity cantilevered walls, but derive additional lateral resistance from ground anchors (tiebacks), concrete anchors, or pile anchors. These anchors are located behind the potential failure surfaces in the retained soil and are connected to the wall structurally. The method of support and anchorage depends on site conditions, design height, and loading imposed. The cost of these walls is variable depending on earth retaining requirements, site geology, aesthetic consideration, and site restraints, but is generally higher than “Standard Design Walls” for the same wall geometry and loading conditions. Anchored walls may be used to stabilize an unstable site provided that

Table 210.2
Types of Reinforced Earth Slopes and Earth Retaining Systems(1)

EARTH RETAINING SYSTEM	Construction Method(2)	PS&E By	Typical Facing Material	Recommended Maximum Vertical Height, ft	Ability to Tolerate Differential Settlement(3)
Reinforced Earth Slopes					
Reinforced Embankments	BU	District PE	Vegetation/Soil	160	E
Rock/Soil Anchors	TD	District PE	Soil/Rock	130	E
State Designed Earth Retaining Systems with Standard Plans					
Concrete Cantilever Wall, Type 1 & 1A	BU	District PE	Concrete	36, 12, 22(4)	P
Concrete L-Type Cantilever Wall, Type 5	BU	District PE	Concrete	12(4)	P
Concrete Masonry Wall, Type 6	BU	District PE	Masonry	6(4)	P
Crib Wall: Concrete, Steel	BU	District PE	Concrete, Steel	50, 36, (4)	P
State Designed Earth Retaining Systems Which Require Special Designs					
Standard Plan Walls with modified wall geometry, foundations or loading conditions	BU	Structure PE	Concrete, Steel, Timber	50	P-F
Non-Gravity Cantilevered Walls					
Sheet Pile Wall	TD	Structure PE	Steel	20	F
Soldier Pile Wall with Lagging	TD/BU	Structure PE	Concrete, Steel, Timber	20	F-G
Tangent Soldier Pile Wall	TD/BU	Structure PE	Concrete	30	F
Secant Soldier Pile Wall	TD	Structure PE	Concrete	30	F
Slurry Diaphragm Wall	TD	Structure PE	Concrete, Shotcrete	80(5)	F
Deep Soil Mixing Wall	TD	Structure PE	Shotcrete	80(5)	F-G
Anchored Wall (Structural or Ground Anchors)	TD	Structure PE	Concrete, Steel, Timber	80(6)	F-G
Gravity Walls					
Concrete Gravity Wall	BU	Structure PE	Concrete	6	P
Rock Gravity Wall	BU	District PE	Rock	13	E
Gabion Basket Wall	BU	District PE	Wire & Rock	26	E
Soil Reinforcement Systems					
Mechanically Stabilized Embankment	BU	Structure PE	Concrete	50	G
Salvaged Material Retaining Wall	BU	District PE	Steel, Timber	16	G
Soil Nail Wall	TD	Structure PE	Concrete, Shotcrete	80	F
Tire Anchored Timber Wall	BU	District PE	Timber	32	G
Proprietary Earth Retaining Systems (Pre-approved)					
The list of Pre-approved systems is available at the website shown in Index 210.2(3)(c).					
Proprietary Earth Retaining Systems (Pending)					
These systems are under review by DES-SD. For more information, see Index 210.2(3)(d).					
Experimental State Designed Earth Retaining Systems					
Geosynthetic Reinforced Walls	BU	Structure PE/ District PE	Concrete Blocks, Steel, Vegetation, Fabric	65	E
Mortarless Concrete Blocks Gravity Walls	BU	District PE	Concrete Blocks	8	P

NOTES:
1. Comparative cost data is available from DES-SD.
2. BU = Bottom Up; TD = Top Down
3. E = Excellent; G = Good; F = Fair; P = Poor
4. Maximum Design Height
5. Anchors may be required
6. With lagging

adequate material exists at the site for the anchors. Economical wall heights up to 80 feet are feasible.

- Gravity Wall Systems that require special designs are Concrete Gravity, Rock Gravity, and Gabion Basket Walls. Concrete Gravity Walls (PS&E by Structure PE). Concrete gravity walls are most economical at design heights below 4 feet. However, they may be constructed at heights up to 6 feet. These walls can be used in connection with a cantilever wall if long lengths of wall with design heights of less than 4 feet are required.

- Rock Gravity Walls (PS&E by District PE). Rock gravity walls consist of rocks that are 100 pounds to 200 pounds, stacked on top of each other at slight batter. These walls are typically used in areas where a rock appearance is desirable for aesthetic reasons. Wall heights range from 1 foot 6 inches to 15 feet, but are most economical for heights less than 10 feet.

- Gabion Basket Walls (PS&E by District PE). Gabion basket walls use compartmented units filled with stones and can be constructed up to 26 feet in height. Each unit is a rectangular basket made of galvanized steel wire. The stone fill is 4 inches to 16 inches in size. Gabion basket walls are typically used for soil and stream bank stabilization. Service life of the gabion basket wall is highly dependent on the environment in which they are placed. Corrosion, abrasion, rock impact, fire and vandalism are examples of site-specific factors that would influence the service life of the wall and should be taken into consideration by the District PE during the design of the project. See Standard Plans for further details.

- Soil Reinforcement Systems. Soil reinforcement systems consist of facing elements and soil reinforcing elements incorporated into a compacted or in situ soil mass. The reinforced soil mass functions similar to a gravity wall.

  Soil reinforcing elements can be any material that provides tensile strength and pullout resistance, and possesses satisfactory creep characteristics and service life. Generally, reinforcing elements are steel, but polymeric and fiberglass systems may be used.

  Facing elements for most systems are either reinforced concrete, light gauge steel, or treated wood. Polymeric reinforced walls may be faced with masonry-like elements or even planted with local vegetation. Selection of facing type is governed by aesthetics and service life.

  Special details are required when drainage structures, overhead sign supports or noise barriers on piles are within the reinforced soil mass. Concrete traffic barriers require a special design support slab when used at the top of the facing of these systems. These systems cannot be used where site restrictions do not allow necessary excavation or placement of the soil reinforcing elements.

  Soil reinforcement systems that require special design are as follows:

- Mechanically Stabilized Embankment (MSE) (PS&E by Structure PE). This system uses welded steel wire mats, steel strips or polymeric materials as soil reinforcing elements. The facing elements are precast concrete. In many cases, this system can be constructed using on-site backfill materials.

  When the bottom-up construction method is possible and other conditions permit their use, these systems are generally the most economical choice for wall heights greater than 20 feet. They may also be the most economical system for wall heights in the 10-foot to 20-foot range, depending on the specific project requirements.

Because of the articulated nature of the facing elements these systems use, they can tolerate greater differential settlement than can monolithic conventional rigid retaining walls, such as concrete cantilever retaining walls.

Steel elements used in this method are sized to provide sacrificial steel to compensate for anticipated corrosion; and may be galvanized to provide additional protection.

- Salvaged Material Retaining Wall (PS&E by District PE). This system utilizes C-channel sections as soil reinforcement. Galvanized metal beam guardrail elements, timber posts or concrete panels are used as facing elements. Often these materials can be salvaged from projects. The District Recycle Coordinator should be consulted as to the availability of salvaged materials.

- Soil Nail Wall (PS&E by Structure PE). This system reinforces either the original ground or an existing embankment during the excavation process. Soil nailing is always accomplished from the top-down in stages that are typically 4 feet to 6 feet in height. After each stage of excavation, corrosion protected soil reinforcing elements, "soil nails", are placed and grouted into holes which have been drilled at angles into the in situ material. The face of each stage of excavation is protected by a layer of reinforced shotcrete. After the full height of wall has been excavated and reinforced, a finish layer of concrete facing is placed either by the shotcreting method or by casting within a face form.

  When top-down construction is possible and conditions permit its use, soil nail wall systems are generally the most economical choice for wall heights greater than 10 feet. Wall heights in excess of 80 feet are feasible in specific locations.

  Because soil nailing is accomplished concurrent with excavation, and thus results in an unloading of the foundation, there is typically no significant differential settlement.

  Steel "soil nails" used in this method are protected against corrosion either by being epoxy coated or encapsulated within a grout filled corrugated plastic sheath, and surrounded by portland cement grout placed during construction. Soil nail lengths typically range from 80 to 100 percent of the wall height, the actual length depends on the nail spacing used and the competency of the in situ soil.

- Recycled Tire Anchor Timber (TAT) Walls (PS&E by District PE). This system utilizes steel bars with recycled tire sidewalls attached by cross bars as soil reinforcing elements. The facing elements are treated timber. TAT walls have a rustic appearance, which makes them suitable in rural environments. The length of commercially available timber post generally controls the height of wall but heights up to 32 feet are feasible.

(c) Proprietary Earth Retaining Systems (Pre-approved).

These conventional retaining walls, cribwalls, and soil reinforcement systems are designed, manufactured, and marketed by vendors. These systems are termed “proprietary” because they are patented.

"Pre-approval" status means that these systems may be listed in the Special Provisions of the project as an Alternative Earth Retaining System (AERS), see Index 210.3, when considered appropriate for a particular location. For a proprietary system to be given "pre-approval" status, the vendor must submit standard plans and design calculations to the Division of Engineering Services – Structure Design (DES-SD) for their review and approval. The Proprietary earth retaining systems that have been pre-approved are included in the Department's Pre-Qualified Products List, located on the following website: www.dot.ca.gov/hq/esc/approved_products_list/ .

Design details and specifications of "pre-approved" proprietary earth retaining systems may be found on the vendor websites listed in the Pre-Qualified Products List. New systems are added to the website list once they are pre-approved for use.

(d) Proprietary Earth Retaining Systems (Pending).

The systems in this category have been submitted by vendors to DES-SD for evaluation. Upon approval of DES-SD, pending systems are added to the website list of "pre-approved" proprietary earth retaining systems and included in the project specific Special Provisions.

If a proprietary system is the only retaining system deemed appropriate for use at a specific location, the construction of that system must be justified or designated an experimental construction feature in accordance with existing Departmental Policy concerning sole source purchases. See Index 110.10 for additional guidance on the use of proprietary items.

(e) Experimental State Designed Earth Retaining Systems.

Every earth retaining system is evaluated before being approved for routine use by the Department. Newly introduced designs, unproven combinations of proprietary and non-proprietary designs or products, are considered experimental. Once an experimental system has been evaluated and approved, it will be made available for routine use. The use of these systems is only permitted upon consultation with the Division of Engineering Services – Geotechnical Services (DES-GS).

Some earth retaining systems which are currently considered experimental follow:

- Geosynthetic Reinforced Walls (PS&E by District PE). These systems utilize geosynthetic material as the soil reinforcing elements. The face of these walls can be left exposed if the geosynthetic material has been treated to prevent decay from ultra-violet rays. Concrete panels, mortarless masonry, tar emulsion, or air blown mortar may be used as facing materials or the face may be seeded if a more aesthetic treatment is preferred. Design is by DES-GS.
- Mortarless Concrete Block Gravity Walls (PS&E by District PE). These wall types consist of vertically stacked, dry cast, concrete blocks. This system utilizes the friction and shear developed between the blocks and the combined weight of the blocks to retain the backfill. Some of these walls have been used as erosion protection at abutments and on embankments. They can be used as an aesthetic treatment for geosynthetic material reinforced walls. All of these walls require a batter. Design is by the DES-GS.

210.3 Alternative Earth Retaining Systems (AERS)

Using the Alternative Earth Retaining Systems (AERS) procedure encourages competitive bidding and potentially results in project cost savings. Therefore, AERS must be considered in all projects where earth retaining systems are required.

The AERS procedure may result in one or more earth retaining systems being included in the contract bid package. Under this procedure, a fully detailed State designed earth retaining system will be provided for each location, and will be used as the basis for payment. Additional systems may be presented in the contract documents as alternatives to the fully detailed State design and can be considered for use at specified locations. The fully detailed State designed earth retaining system may be either a Standard Plan system or a special design system. Alternative systems may also be State designed systems, "pre-approved" proprietary systems or experimental systems, as appropriate. The State designed alternative systems, both Standard Plan walls and special design systems, are to be completely designed and specified in the PS&E. Alternative systems are to be listed in the Special Provisions as AERS.

The AERS procedure requires the involvement of the District PE, DES-SD, and the DES-GS. The District PE should submit pertinent site information (site plans, typical sections, etc.) to DES-GS for a feasibility study as early as possible in the project development process.

Under the AERS procedure, parts of the PS&E package which pertain to the earth retaining systems will be prepared as follows:

- Contract plans for State designed systems can be prepared by the District PE (Standard Plan systems), the DES-GS (special design soil reinforcement systems and experimental systems), or the Structure PE (Standard Plan systems and special design systems).
- "Pre-approved" proprietary systems that are determined, based on consultation with DES-SD, to be appropriate alternatives to the State designed earth retaining system, are to be listed in the Special Provisions.
- Specifications and Estimates shall be developed for the fully detailed State designed system, which will be used as the basis for payment.

The earth retaining systems utilizing this procedure are to be measured and paid for by the square yard area of the face of the earth retaining system. Should an AERS be constructed, payment will be made based on the measurements of the State designed system which was designated as the basis of payment. The contract price paid per square yard is for all items of work involved and includes excavation, backfill, drainage system, reinforcing steel, concrete, soil reinforcement, and facing. Any barrier, fence, or railing involved is measured and paid for as separate contract cost items.

210.4 Cost Reduction Incentive Proposals (CRIP)

Sometimes Contractors submit proposals for an earth retaining system under Section 5-1.14 of the Standard Specifications, "Cost Reduction Incentive." The Contractor proposed system may modify or replace the earth retaining system permitted by the contract. The CRIP process allows vendors of proprietary earth retaining systems an alternative method for having their systems used prior to obtaining "pre-approval" (see Index 210.2(c)). CRIP submittals are administered by the Resident Engineer. However, Contract Change Orders are not to be processed until the CRIP is approved by Headquarters Construction with review assistance provided by the District or Structure PE as appropriate.

210.5 Aesthetic Consideration

The profile of the top of wall should be designed to be as pleasing as the site conditions permit. All changes in the slope at the top of cast-in-place concrete walls should be rounded with vertical curves at least 20 feet in length. Abrupt changes in the top of the wall profile should be avoided by using vertical curves, slopes, steps, or combinations thereof. Side slopes may be flattened or other adjustments made to provide a pleasing profile.

Where walls are highly visible, special surface treatments or provisions for landscaping should be considered. The aesthetic treatment of walls should be discussed with the District Landscape Architect and when necessary referred to DES Structure Design Services for additional study by the Office of Transportation Architecture.

The wall area between the grade line and 6 feet above it shall be free of any designed indentations or protrusions that may snag errant vehicles.

When alternative wall types are provided on projects with more than one wall site, any restrictions as to the combination of wall types should be specified in the Special Provisions.

210.6 Safety Railing, Fences, and Concrete Barriers

<u>Cable railing should be installed for employee protection in areas where employees may work adjacent to and above vertical faces of retaining walls, wingwalls, abutments, etc. where the vertical fall is 4 feet or more.</u>

If cable railing is required on a wall which is less than 4 feet 6 inches tall and that wall is located within the clear recovery zone, then the cable railing should be placed behind the wall. See Standard Plan B11-47 for details of cable railing.

Special designs for safety railing may be considered where aesthetic values of the area warrant special treatment. In addition, if the retaining wall is accessible to the public and will have pedestrians or bicycles either above or below the retaining wall, then the provisions of Index 208.10 shall apply.

Concrete barriers may be mounted on top of retaining walls. Details for concrete barriers mounted on top of retaining walls Type 1 through 5 are shown in the Standard Plans. A concrete barrier slab is required if a concrete barrier is to be used at the top of a special design earth retaining system. DES-SD should be contacted for preparation of the plans involved in the special design.

Retaining walls joining right of way fences should be a minimum of 6 feet clear height.

The District PE should examine the proposed retaining wall location in relation to the provisions of Index 309.1 to ensure adequate horizontal clearances to the structure or to determine the type and placement of the appropriate roadside safety devices.

210.7 Design Responsibility

The Structure PE has primary responsibility for the structural design and preparation of the contract documents (PS&E) for special design earth retaining systems involving Standard Plans non-gravity cantilevered walls, anchored walls, concrete and rock gravity walls, mechanically stabilized embankment, and soil nail walls. The DES-GS has primary responsibility for the geotechnical design of all reinforced earth slopes and earth retaining systems. DES-SD will prepare the Specifications and Engineer's Estimate for contracts when the AERS procedure is used. DES-SD reviews and approves standard plan submittals for proprietary earth retaining systems submitted by vendors. DES-SD and DES-GS assist Headquarters Construction in evaluating the CRIP submitted by contractors.

Districts may prepare contract plans, specifications, and engineer's estimate for Standard Plan retaining walls provided the foundation conditions and site requirements permit their use. A foundation investigation is required for all reinforced earth slopes and earth retaining systems. PS&E's for slurry walls, deep soil mixing walls, gabion walls, tire anchored timber walls, salvaged material walls, and experimental walls will be prepared by the District PE with assistance from DES-GS. Earth retaining systems may be included in the PS&E as either highway or structure items.

The time required for DES-SD to provide the special design of a retaining system is site and project dependent. Therefore, the request for a special design should be submitted by the District PE to DES-SD as far in advance as possible, but not less than 6 months prior to PS&E delivery. At least 3 months is required to conduct a foundation investigation for an earth retaining system. A site plan, index map, cross sections, vertical and horizontal alignment, and utility and drainage requirements should be sent along with the request.

DES-GS has the responsibility for preparing a feasibility study for AERS. The District PE should submit project site information (site plans, typical sections, etc.) as early in the planning stage as possible so that determination of the most appropriate earth retaining system to use can be made.

210.8 Guidelines for Type Selection and Plan Preparation

(1) Type Selection. Type selection for reinforced earth slopes and earth retaining systems should be based on considerations set forth in Index 210.2.

The District PE should request a feasibility study for a reinforced slope or earth retaining system from DES-GS as early as possible in the project development process. After the feasibility study, the District PE should request an Advanced Planning Study (APS) from DES-SD for all special design earth retaining systems that DES-SD may be required to include in the PS&E.

If the District PE decides that the course of action favors an earth retaining system in which the PS&E will be delivered by DES-SD, then a Bridge Site Data Submittal – Non-Standard Retaining Wall/Noise Barrier must be submitted to DES-Structure Design Services & Earthquake Engineering – Preliminary Investigations (PI) Branch. A copy of this submittal will be forwarded to DES-SD and DES-GS by PI.

The Structure PE, with input from DES-GS and the District PE, will then type select the appropriate earth retaining system for the site and project. After an earth retaining system has been type selected, then DES-GS will prepare a Geotechnical Design Report.

The process for type selecting and developing the PS&E for reinforced earth slopes and earth retaining systems is set forth in Figure 210.8.

All appropriate State designed and proprietary earth retaining systems should be considered for inclusion in the contract documents to promote competitive bidding, which can result in cost savings.

(2) Foundation Investigations. DES-GS should be requested to provide a foundation recommendation for all sites involving a reinforced slope or an earth retaining system. Any log of test boring sheets accompanying the foundation reports must be included with the contract plans as project information, for the bidders use.

(3) Earth Retaining Systems with Standard Plans. The following guidelines should be used to prepare the contract plans for earth retaining systems, which are found in the Standard Plans:

(a) Loads. All wall types selected must be capable of supporting the field surcharge conditions. The design surcharges can be found in the Standard Plans. Deviance from these loadings will require a special design

(b) Footing Steps. For economy and ease of construction of wall Types 1 through 6, the following criteria should be used for layout of footing steps.

- Distance between steps should be in multiples of 8 feet.
- A minimum number of steps should be used even if a slightly higher wall is necessary. Small steps, less than 1 foot in height, should be avoided unless the distance between steps is 96 feet or more. The maximum height of steps should be held to 4 feet. If the footing thickness changes between steps, the bottom of footing elevation should be adjusted so that the top of footing remains at the same elevation.

(c) Sloping Footings. The following criteria should be used for layout of sloping footings.

- The maximum permissible slope for reinforced concrete retaining walls is 3 percent. Maximum footing slope for masonry walls is 2 percent.
- When sloping footings are used, form and joint lines are permitted to be perpendicular and parallel to the footing for ease of construction.
- In cases where vertical electroliers or fence posts are required on top of a wall, the form and joint lines must

also be vertical. A sloping footing should not be used in this situation since efficiency of construction would be lost.

Sloping footing grades should be constant for the entire length of the wall. Breaks in footing grade will complicate forming and result in loss of economy. If breaks in footing grade are necessary, a level stepped footing should be used for the entire wall.

- When the top of wall profile of crib walls is constant for the entire length, the bottom of wall profile may be sloped to avoid steps in the top of wall. In this case, all steps to compensate for changes of wall height and original ground profile would be made in the bottom of wall. The maximum permissible slope is 6 percent. If vertical electroliers or fence posts are required on top of the wall, the crib wall should not be sloped. Sloping crib walls are permissible with guard railing with vertical posts.

(d) Wall Joints. General details for required wall joints on wall Types 1, 1A, 2, and 5 are shown on Standard Plan B0-3. Expansion joints, Bridge Detail 3-3, should be shown at maximum intervals of 96 feet. Shorter spaces should be in multiples of 8 feet. Expansion joints generally should be placed near angle points in the wall alignment. When concrete barriers are used on top of retaining walls, the waterstop in the expansion joint must be extended 6 inches into the barrier. This detail should be shown or noted on the wall plans. Weakened plane joints, Bridge Detail 3-2, should be shown at nearly equal spaces between joints.

(e) Drainage. Gutters should be used behind walls in areas where it is necessary to carry off surface water or to prevent scour. Low points in wall vertical alignment or areas between return walls must be drained by downspouts passing through the walls. Standard Plan B3-9 shows typical drainage details. Special design of surface water drainage facilities may be necessary depending on the amount of surface water anticipated. Where ground water is likely to occur in any quantity, special provisions must be made to intercept the flow to prevent inundation of the backfill and unsightly continuous flow through weep holes.

(f) Quantities. When the AERS procedure is not utilized, quantities for each wall item of work are usually developed for payment. The quantities for concrete, expansion joint waterstop, structure excavation, structure backfill, pervious backfill material, concrete barrier or railing, and gutter concrete must also be tabulated. Quantities should be tabulated on the plans for each wall.

(4) Soil Reinforcement Systems. The following guidelines should be used to prepare the contract plans for soil reinforcement systems:

(a) Leveling Pads. Most soil reinforcement systems do not require extensive foundation preparation. It may be necessary, however, to design a concrete leveling pad on which to construct the face elements. A reinforced concrete leveling pad will be required in areas prone to consolidation or frost disturbance.

- Steps in the leveling pad should be the same height as the height of the facing elements or thickness of the soil layer between the soil reinforcement.
- Distance between steps in the leveling pad should be in increments equivalent to the length of individual facing elements.
- A minimum number of steps should be used even if a slightly higher wall is necessary.

(b) Drainage. Gutters should be used behind walls in areas where it is necessary to carry off surface water or to prevent scour. Low points in wall vertical alignment or areas between return walls must be drained by downspouts passing through the walls. Special design of surface water drainage facilities will be necessary and should be prepared by DES-SD. Where ground water is likely to occur in any quantity, special provisions must be made to intercept the flow to prevent inundation of the backfill.

(c) Quantities. When the AERS procedure is not utilized, quantities for each item of work are usually developed for payment. Bid items must include, but not be limited to: excavation and backfill for the embedment depth, soil reinforcement, facing elements, and concrete for leveling pad construction. Additional bid items for inclusion are any drainage system, pervious backfill, concrete barrier, railings, and concrete gutters. Quantities should be tabulated on the plans for each wall.

(5) Earth Retaining Systems. The following miscellaneous details are applicable to all earth retaining systems:

(a) Utilities. Provisions must be made to relocate or otherwise accommodate utilities conflicting with the retaining wall. A utility opening for a Type 1 wall is shown on Standard Plan B3-9. Any other utility openings will require special design details and should be reviewed by DES-SD.

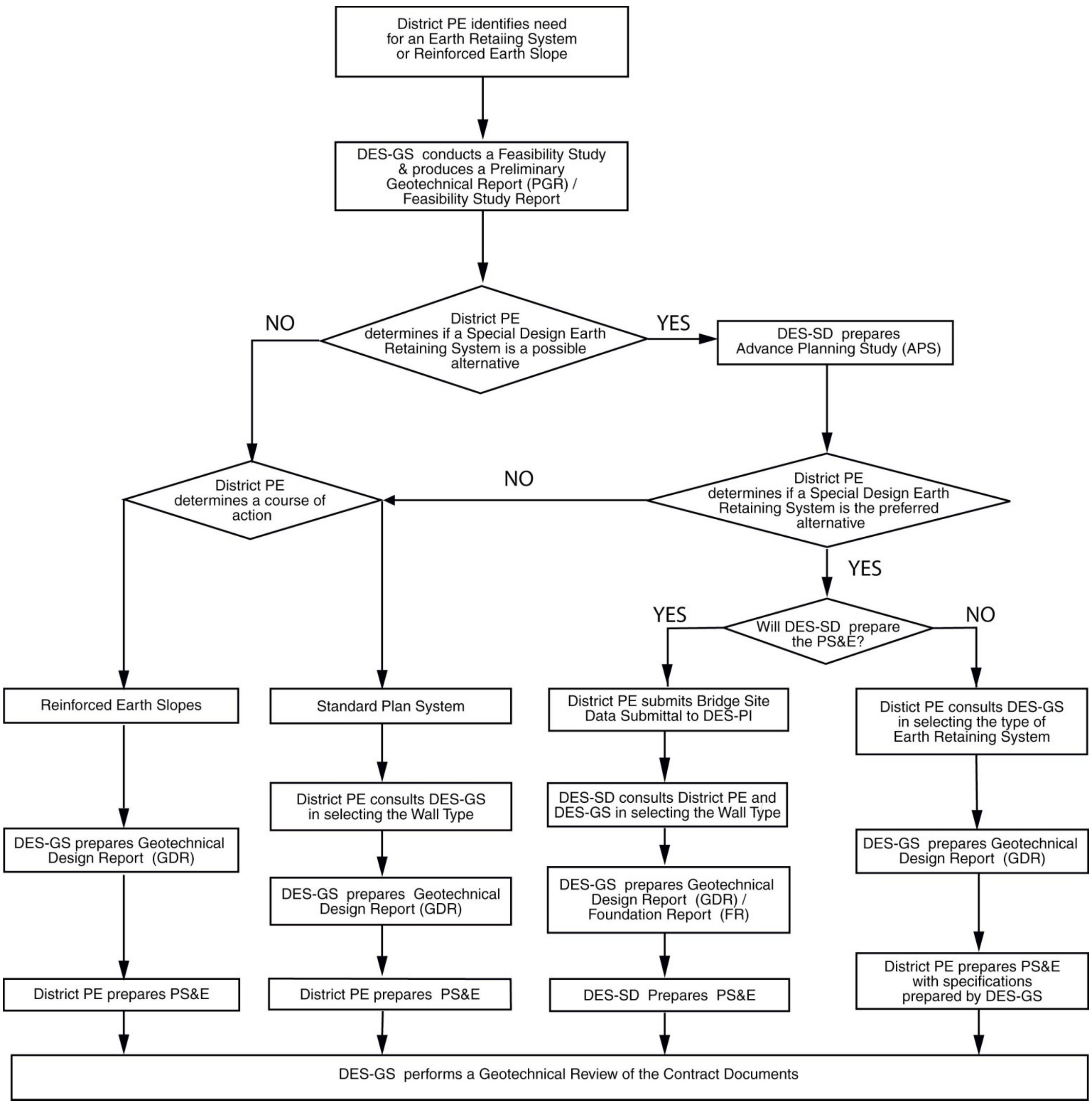
(b) Electroliers and Signs. Details for mounting electroliers and signs on earth retaining systems are designed by DES-SD. Requests for preparation of details should be made at least 3 months in advance of the PS&E submittal to District Officer Engineer date. To accommodate the base plates for overhead signs, a local enlargement may affect the horizontal clearance to both the edge of pavement and the right of way line. This type of enlargement should be considered at the time of establishing the wall layout and a need for a Mandatory Design Exception determined. For mounting details, furnish DES-SD a complete cross section of the roadway at the sign and the layout and profile of the earth retaining system.

(c) Fence and Railing Post Pockets. Post pocket details shown for cable railing in the Standard Plans may also be used for mounting chain link fence on top of retaining walls. Special details may be necessary to accommodate the reinforcement in soil reinforcement systems.

(d) Return Walls. Return walls should be considered for use on the ends of the walls to provide a finished appearance. Return walls are necessary when wall offsets are used or when the top of wall is stepped. Return walls for soil reinforcement systems will require special designs to accommodate the overlapping of soil reinforcing elements.

All special wall details such as sign bases, utility openings, drainage features, fences, and concrete barriers should be shown on the plan sheet of the wall concerned or included on a separate sheet with the wall plan sheets. Details should be cross-referenced on the wall sheets to the sheets on which they are shown.

Figure 210.8
Type Selection and PS&E Process for Reinforced Earth Slopes and Earth Retaining Systems



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, 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.** A buffered bike lane may also be established within the roadbed, separated by a marked buffer between the bike lane and the traffic lane or parking lane. See the California MUTCD for further buffered bike lane marking and signing guidance. Contraflow bike lanes are designed for bike travel in the opposite direction as adjacent vehicular traffic, and are only allowed on one-way streets. See the California MUTCD for contraflow bike lane marking and signing guidance. 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 a Class IV bikeway (separated bikeway). See DIB 89 for Class IV bikeway (separated bikeway) design guidance. **The minimum Class II bike lane width shall be 4 feet, except where:**

   - <u>Adjacent to on-street parking, the minimum bike lane should be 5 feet.</u>
   - <u>Posted speeds are greater than 40 miles per hour, the minimum bike lane should be 6 feet, or</u>
   - **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.

<u>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.</u> **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 (tent 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 new construction as well as pavement overlay projects.

At freeway entrances and exits, the maximum difference in cross slope between adjacent lanes, or between lanes and gore areas, should not exceed 5 percent.

Topic 302 - Highway Shoulder Standards

302.1 Width

The shoulder widths given in Table 302.1 shall be the minimum continuous usable width of paved shoulder on highways. Typically, on-street parking areas in urbanized areas is included in the shoulder.

Class II bikeways are typically part of the shoulder width, see Index 301.2. **Where rumble strips are placed in the shoulder, the shoulder shall be a minimum of 4 feet width to the right of the grooved rumble strip when a vertical element, such as curb or guardrails present or a minimum of 3 feet width when a vertical element is not present.** Shoulder rumble strip must not be placed in the Class II bike lane. Consult the District Traffic Safety Engineer during selection of rumble strip options and with the California MUTCD for markings in combination with rumble strip. Also see Standard Plans for rumble strip details.

See Design Information Bulletin Number 79, for 2R, 3R, certain storm damage, protective betterment, operational, and safety projects on two-lane conventional highways and three-lane conventional highways.

See Index 308.1 for shoulder width requirements on city streets or county roads. See shoulder definition, Index 62.1(8).

See Index 1102.2 for shoulder width requirements next to noise Barriers.

When shoulders are less than standard width, see Index 204.5(4) for bicycle turnout considerations.

302.2 Cross Slopes

(1) *General* - **When a roadway crosses a bridge structure, the shoulders shall be in the same plane as the adjacent traveled way**.

(2) *Left Shoulders* - **In depressed median sections, shoulders to the left of traffic shall be sloped at 2 percent away from the traveled way.**

In paved median sections, shoulders to the left of traffic shall be designed in the plane of the traveled way. Maintenance paving beyond the edge of shoulder should be treated as appropriate for the site, but consideration needs to be given to the added runoff and the increased water depth on the pavement (see discussion in Index 831.4(5) "Hydroplaning").

(3) *Right Shoulders*- In normal tangent sections, **shoulders to the right of traffic shall be sloped at 2 percent to 5 percent away from the traveled way.**

The above flexibility in the design of the right shoulder allows the designer the ability to conform to regional needs. Designers shall consider the following during shoulder cross slope design:

- In most areas a 5 percent right shoulder cross slope is desired to most expeditiously remove water from the pavement and to allow gutters to carry a maximum water volume between drainage inlets. The shoulders must have adequate drainage interception to control the "water spread" as discussed in Table 831.3 and Index 831.4. Conveyance of water from the total area transferring drainage and rainwater across each lane and the quantity of intercepting drainage shall also be a consideration in the selection of shoulder cross slope. Hydroplaning is discussed in Index 831.4 (5).
- In locations with snow removal operations it is desirable for right shoulders to slope

Table 302.1
Mandatory Standards for Paved Shoulder Widths on Highways

Highway Type	Paved Shoulder Width (ft) Left	Right [8]
Freeways & Expressways		
2 lanes [1]	--	8[6]
4 lanes [1]	5	10
6 or more lanes [1]	10	10
Auxiliary lanes	--	10
Freeway-to-freeway connections		
Single and two-lane connections	5	10
Three-lane connections	10	10
Single-lane ramps	4[2]	8
Multilane ramps	4[2]	8[3]
Multilane undivided	--	10
Collector-Distributor	5	10
Conventional Highways		
Multilane divided		
4-lanes	5	8[7]
6-lanes or more	8	8[7]
Urban areas with posted speeds less than or equal to 45 mph and curbed medians	2[4]	8[7]
Multilane undivided	--	8[7]
2-lane		
RRR	See Index 307.3	
New construction	See Table 307.2	
Slow-moving vehicle lane	--	4[5]
Local Facilities		
Frontage roads	See Index 310.1	
Local facilities crossing State facilities	See Index 308.1	

NOTES:

(1) Total number of lanes in both directions including separate roadways (see Index 305.6). If a lane is added to one side of a 4-lane facility (such as a truck climbing lane) then that side shall have 10 feet left and right shoulders. See Index 62.1.

(2) May be reduced to 2 feet upon concurrence from the Project Delivery Coordinator that a restrictive situation exists. 4 feet preferred in urban areas and/or when ramp is metered. See Index 504.3.

(3) May be reduced to 2 feet or 4 feet(4 feet preferred in urban areas) in the 2-lane section of a non-metered ramp, which transitions from a single lane upon concurrence from the Project Delivery Coordinator that a restrictive situation exists. May be reduced to 2 feet in ramp sections having 3 or more lanes. See Index 504.3(b).

(4) For posted speeds less than or equal to 35 mph, shoulder may be omitted (see Index 303.5(5)) except where drainage flows toward the curbed median.

(5) On right side of climbing or passing lane section only. See Index 301.2(1) for minimum width if bike lanes are present.

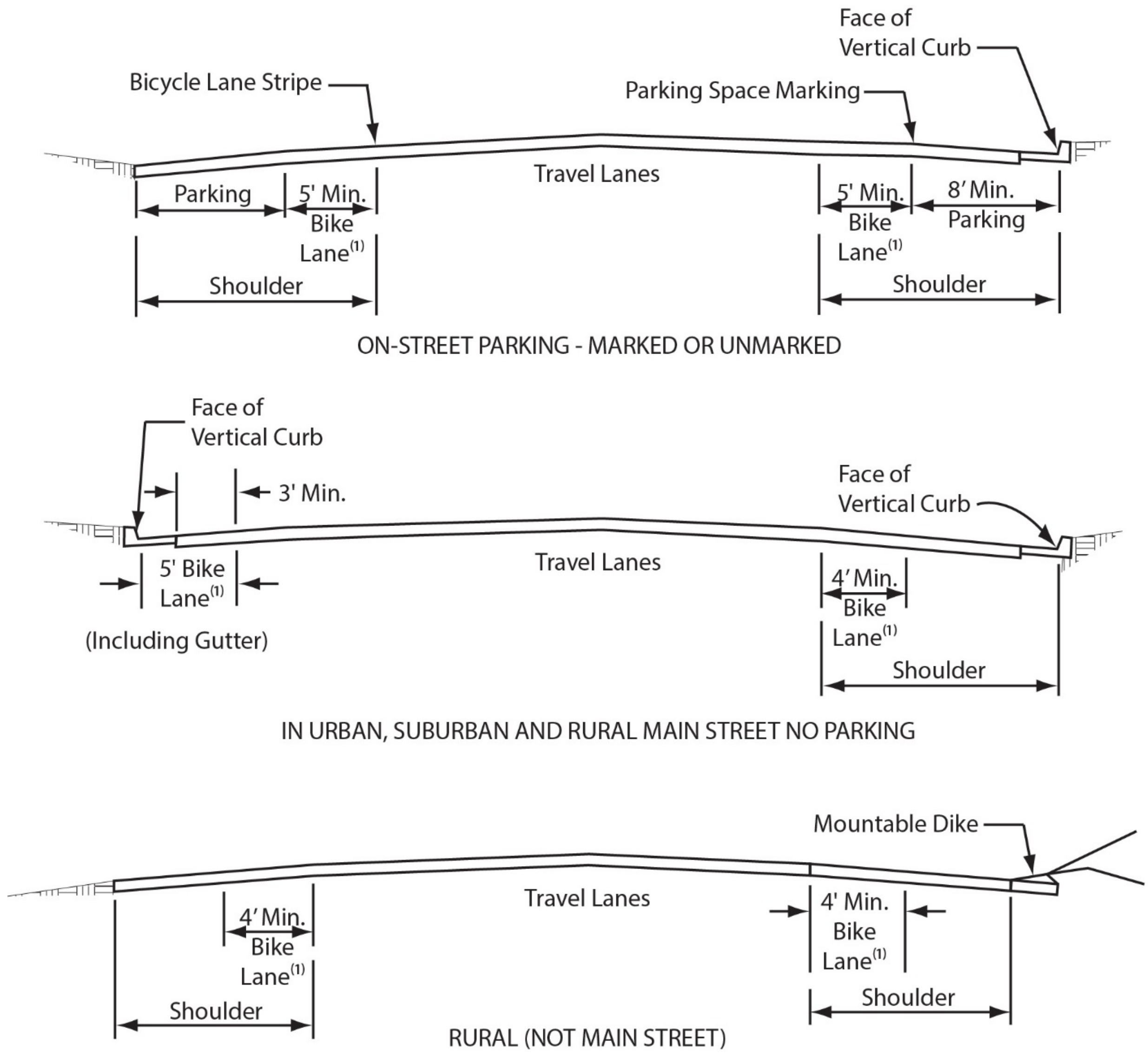
(6) 10-foot shoulders preferred.

(7) Where on-street parking is allowed, 10 feet shoulder width is preferred. Where bus stops are present, 10 feet shoulder width is preferred for the length of the bus stop. If a Class II bikeway is present, minimum shoulder width shall be 8 feet where on street parking is provided plus the minimum required width for the bike lane.

(8) Shoulders adjacent to abutment walls, retaining walls in cut locations, and noise barriers shall be not less than 10 feet wide. See Index 303.4 for minimum shoulder adjacent to bulbouts.

Figure 301.2A

Typical Class II Bikeway (Bike Lane) Cross Sections



NOTES:

(1) See Index 301.2 for additional guidance.

(2) For pavement marking guidance, see the California MUTCD, Section 9C.04.

away from traffic in the same plane as the traveled way. This design permits the snowplowing crew to remove snow fromthe lanes and the shoulders with the least number of passes.

- For 2-lane roads with 4-foot shoulders, see Index 307.2.
- If shoulders are Portland cement concrete and the District plans to convert shoulders into through lanes within the 20 years following construction, then shoulders are to be built in the plane of the traveled way and to lane standards for width and structural section. (See Index 603.4).
- Deciding to construct pedestrian facilities and elements, where none exist, is an important consideration. Shoulders are not required to be designed as accessible pedestrian routes although it is legal for a pedestrian to traverse along a highway. In urban, rural main street areas, or near schools and bus stops with pedestrians present, pedestrian facilities should be constructed. In rural areas where few or no pedestrians exist, it would not be reasonable or cost effective to construct pedestrian facilities. This determination should involve the local agency and must be consistent with the design guidance provided in Topic 105 and in Design Information Bulletin 82, "Pedestrian Accessibility Guidelines for Highway Projects" for people with disabilities.

Shoulder slopes for superelevated curves are discussed in Index 202.2.

See Index 307.2 for shoulder slopes on 2-lane roads with 4-foot shoulders.

302.3 Tapered Edge

The tapered edge is a sloped edge that is placed at the edge of the paved roadbed to provide a smooth reentry for vehicles that leave the roadway. Its design is based on research performed by the FHWA.

The tapered edge is placed on all traversable pavement edges either during new construction or on overlay projects irrespective of pavement types and is most useful:

- On undivided roadways.
- On roadways with unpaved shoulders or where paved shoulders width is less than 5 feet.
- On roadways with Class II Bikeways.

The tapered edge is not to be placed on roadways:

- Next to curbs, dikes, guardrails, barriers, walls, right-turn lanes, accelerations lanes and landscape paving.
- Where the distance from the edge of the paved roadbed to the hinge point is less than 1 foot and there is not enough room to place the tapered edge.
- Within 3 feet of driveways or intersections.
- Where pavement overlay thickness is less than 0.15 foot.

Tapered edge is optional when the distance between consecutive minor roads or driveways is less than 30 feet. See the Standard Plans for design and construction details regarding tapered edge.

Topic 303 - Curbs, Dikes, and Side Gutters

303.1 General Policy

Curb (including curb with gutter pan), dike, and side gutter all serve specific purposes in the design of the roadway cross section. Curb is primarily used for channelization, access control, separation between pedestrians and vehicles, and to enhance delineation. Dike is specifically intended for drainage and erosion control where stormwater runoff cannot be cost effectively conveyed beyond the pavement by other means. Curb with gutter pan serves the purpose of both curb and dike. Side gutters are intended to prevent runoff from a cut slope on the high side of a superelevated roadway from running across the pavement and is discussed further in Index 834.3.

Aside from their positive aspects in performing certain functions, curbs and dikes can have undesirable effects. In general, curbs and dikes should present the least potential obstruction, yet perform their intended function. As operating speeds increase, lower curb and dike height is desirable. Curbs and dikes are not considered traffic barriers.

On urban conventional highways where right of way is costly and/or difficult to acquire, it is appropriate to consider the use of a “closed” highway cross section with curb, or curb with gutter pan. There are also some situations where curb is appropriate in freeway settings. The following criteria describe typical situations where curb or curb with gutter pan may be appropriate:

(a) Where needed for channelization, delineation, or other means of improving traffic flow and safety.

(b) At ramp connections with local streets for the delineation of pedestrians walkways and continuity of construction at a local facility.

(c) As a replacement of existing curb with gutter pan and sidewalk.

(d) On frontage roads on the side adjacent to the freeway to deter vehicular damage to the freeway fence.

(e) When appropriate to conform to local arterial street standards.

(f) Where it may be necessary to solve or mitigate operational deficiencies through control or restriction of access of traffic movements to abutting properties or traveled ways.

(g) In freeway entrance ramp gore areas (at the inlet nose) when the gore cross slope exceeds standards.

(h) At separation islands between a freeway and a collector-distributor to provide a positive separation between mainline traffic and collector-distributor traffic.

(i) Where sidewalk is appropriate.

(j) To deter vehicular damage of traffic signal standards.

Dike is appropriate where controlling drainage is not feasible via sheet flow or where it is necessary to contain/direct runoff to interception devices. On cut slopes, dike also protects the toe of slope from erosion. Dike may also be necessary to protect adjacent areas from flooding.

<u>The use of curb should be avoided on facilities with posted speeds greater than or equal to 40 miles per hour, except as noted in Table 303.1. For projects where the use of curb is appropriate, it should be the type shown in Table 303.1.</u>

303.2 Curb Types and Uses

Depending on their intended function, one of two general classifications of curb design is selected as appropriate. The two general classifications are vertical and sloped. Vertical curbs are nearly vertical (approximate batter of 1:4) and vary in height from 4 inches to 8 inches. Sloped curbs (approximate batter of 2:3 or flatter) vary in height from 3 inches to 6 inches.

Sloped curbs are more easily mounted by motor vehicles than vertical curbs. Since curbs are not generally adequate to prevent a vehicle from leaving the roadway, a suitable traffic barrier should be provided where redirection of vehicles is needed. A curb may be placed to discourage vehicles from intentionally entering the area behind the curb (e.g., truck offtracking). In most cases, the curb will not prevent an errant vehicle from mounting the curb.

Curb with gutter pan may be provided to enhance the visibility of the curb and thus improve delineation. This is most effective where the adjacent pavement is a contrasting color or material. B2-4 and B4 curbs are appropriate for enhancing delineation. Where curb with gutter pan is intended as delineation and has no drainage function, the gutter pan should be in the same plane as the adjacent pavement.

The curb sections provided on the Standard Plans are approved types to be used as stated below. The following types are vertical curb, (for information on side gutters, see Index 834.3):

(1) *Types A1-6, A2-6, and A3-6.* These curbs are 6 inches high. Their main function is to provide a more positive deterrent to vehicles than provided by sloped curbs. Specifically, these curbs are used to separate pedestrians from vehicles, to control parking of vehicles, and to deter vehicular damage of traffic signal standards. They may also be used as raised median islands in low speed environments (posted speed $\leq$ 35 miles per hour). These curbs do not constitute a barrier as they can be

Table 303.1

Selection of Curb Type

Location	Posted Speeds (mph)		
	≤ 35	40	≥ 45
Freeways and Expressways			
Collector-distributor Roads	See Index 504.3(11)		
Ramps			
Conventional Highways			
- Frontage Roads (1)	A or B-6	B-6	B-4
- Traffic Signals	A or B-6	B-6	B-4
- Raised Traffic, Median Islands & Pedestrian Refuge Islands (2)	A or B-6	B-6	B-4 or D
- Adjacent to Sidewalks	A (3)	A-6	B-6
- Bulbouts/curb extensions	A	NA	NA
- Bridges (4)	H, A3, or B3	H or B3	B3

NOTES:

(1) Based on the posted speed along the frontage road.

(2) See the National Cooperative Highway Research Program Report 672 entitled "Roundabouts: An Informational Guide, 2nd ed." for information on curbs at roundabouts.

(3) Type A curb includes Types A1-6, A2-6, A1-8, and A2-8.

(4) Type H curb typically used in conjunction with Type A curbs next to sidewalks on approach roadway. Type A3 curbs typically used with corresponding Type A curbs on median island of approach roadway. Type B3 curbs typically used with corresponding Type B curbs on approach roadway.

mounted except at low speeds and flat angles of approach.

(2) *Types A1-8, A2-8, and A3-8.* These 8-inch high curbs may be used in lieu of 6-inch curbs when requested by local authorities, if the curb criteria stated under Index 303.1 are satisfied and posted speeds are 35 miles per hour or less. This type of curb may impede curbside passenger loading and may make it more difficult to comply with curb ramp design (see Design Information Bulletin Number 82, "Pedestrian Accessibility Guidelines for Highway Projects").

(3) *Type H Curb.* This type may be used on bridges where posted speeds are 40 miles per hour or less and where it is desired to match the approach roadway curb. Type H curb is often incorporated into bridge barrier/sidewalk combination railings (See Index 208.10(4)).

These types are sloped curbs:

(4) *Types B1, B2, and B3 Curbs* Types B1-6, B2-6, and B3-6 are 6 inches high. Type B1-4, B2-4, and B3-4 are 4 inches high. Since all have a 1:1½ slope or flatter on the face, they are mounted more easily than Type A curbs. Typical uses of these curbs are for channelization including raised median islands. B2 curb with gutter pan also serves as drainage control.

(5) *Type B4 Curb.* Type B4 curb with gutter pan is 3 inches high and is typically used on ramp gores as described in Index 504.3(11). It may also be appropriate where a lower curb is desirable.

(6) *Type D Curb.* Type D curb is 4 inches or 6 inches high and is typically used for raised traffic islands, collector-distributor separation islands, or raised medians when posted speeds equal or exceed 45 miles per hour.

(7) *Type E Curb*. This essentially is a rolled gutter used only in special drainage situations.

Curbs with gutter pans, along with the shoulder, may provide the principal drainage system for the roadway. Inlets are provided in the gutter pan or curb, or both.

Gutter pans are typically 2 feet wide but may be 1 foot to 4 feet in width, with a cross slope of typically 8.33 percent to increase the hydraulic capacity. Gutter pan cross slopes often need to be modified at curb ramps in order to meet accessibility requirements. See Design Information Bulletin Number 82, "Pedestrian Accessibility Guidelines for Highway Projects" for accessibility standards. Warping of the gutter pan should be limited to the portion within 2 feet to 3 feet of the gutter flow line to minimize adverse driving effects.

Curbs and gutter pans are cross section elements considered entirely outside the traveled way, see Index 301.1.

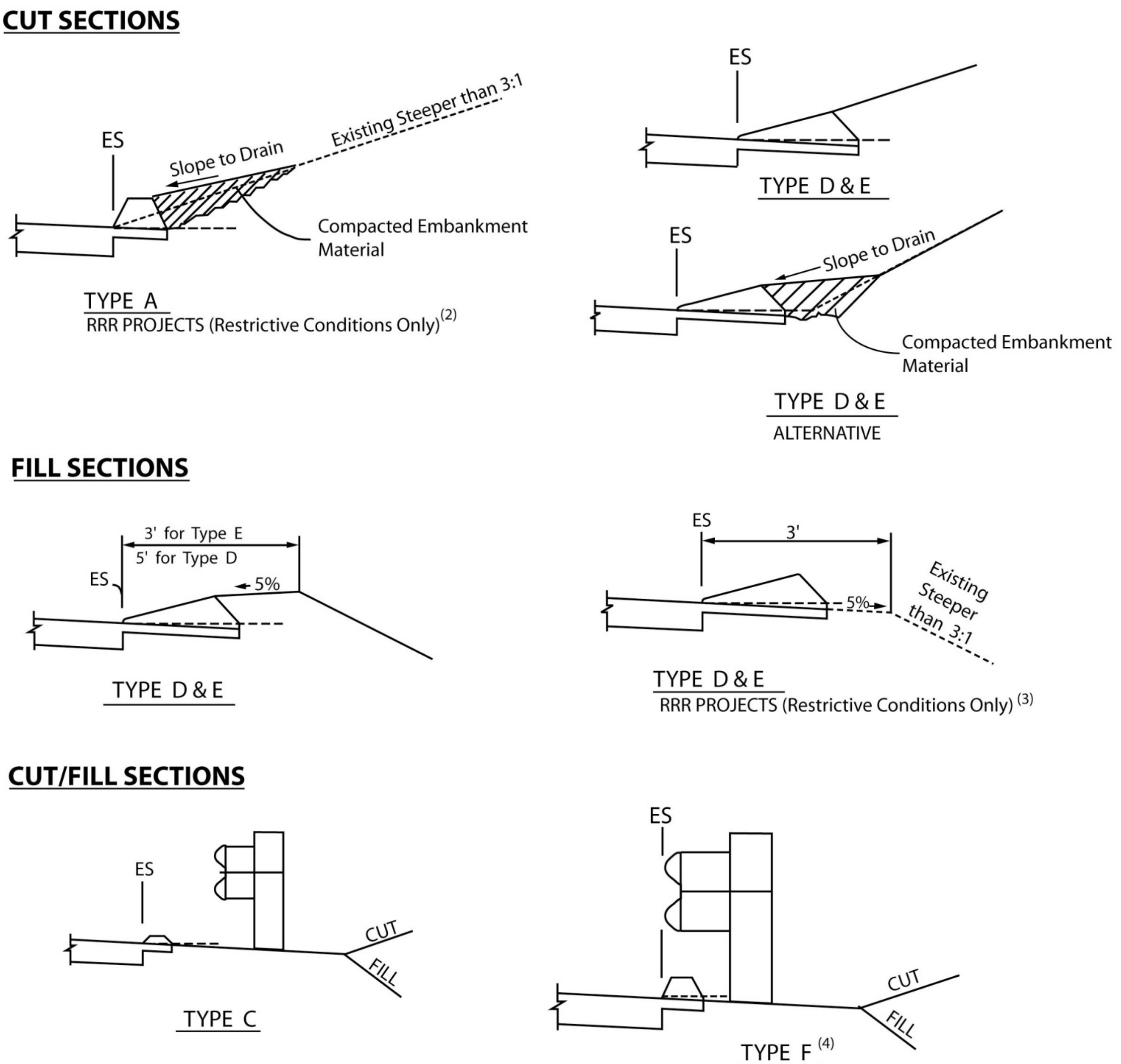
303.3 Dike Types and Uses

Use of dike is intended for drainage control and should not be used in place of curb. Dikes placed adjoining the shoulder, as shown in Figures 307.2, 307.4, and 307.5, provide a paved triangular gutter within the shoulder area. The dike sections provided on the Standard Plans are approved types to be used as stated below. <u>Dikes should be selected as illustrated in Figure 303.3.</u> Dikes should be designed so that roadway runoff is contained within the limits specified in Index 831.3. For most situations Type E dike is the preferred dike type as discussed below.

(1) *Type A Dike.* The use of Type A dike should be avoided. For RRR projects, Type A dike may be used in cut sections with slopes steeper than 3:1 and where existing conditions do not allow for construction of the wider Type D or E dikes. Compacted embankment material should be placed behind the back of dike as shown in Figure 303.3.

(2) *Type C Dike.* This low dike, 2 inches in height, may be used to confine small concentrations of runoff. The capacity of the shoulder gutter formed by this dike is small. Due to this limited capacity, the need for installing an inlet immediately upstream of the beginning of this dike type should be evaluated. This low dike can be traversed by a vehicle and allows the area beyond the surfaced shoulder to be used as an emergency recovery and parking area. The Type C dike is the only dike that may be used in front of

Figure 303.3
Dike Type Selection and Placement(1)



Notes:
- **(1) See Standard Plans for additional information and details.**
- **(2) See Index 303.3(1) for restrictive conditions.**
- **(3) See Index 303.3(3) and Index 303.3(4) for restrictive conditions for Type D and Type E respectively.**
- **(4) Use under MBGR when dike is necessary for drainage control.**

guardrail. In such cases, it is not necessary to place compacted embankment material behind Type C dike.

(3) *Type D Dike.* This 6-inch high dike provides about the same capacity as the Type A dike but has the same shape as the Type E dike. The quantity of material in the Type D dike ismore than twice that of a Type E dike. It should only be used where there is a need to contain higher volumes of drainage. Compacted embankment material should be placed behind the back of dike as shown in Figure 303.3. For RRR projects that do not widen pavement, compacted embankment material may be omitted on existing fill slopes steeper than 3:1 when there is insufficient room to place the embankment material.

(4) *Type E Dike.* This 4-inch high dike provides more capacity than the Type C dike. Because Type E dike is easier to construct than Type D dike, and has greater drainage capacity than Type C dike, it is the preferred dike type for most installations. Compacted embankment material should be placed behind the back of dike as shown in Figure 303.3. For RRR projects that do not widen pavement, compacted embankment material may be omitted on existing fill slopes steeper than 3:1 where there is insufficient room to place the embankment material.

(5) *Type F Dike.* This 4-inch high dike is to be used where dike is necessary for drainage underneath a guardrail installation. This dike is placed directly under the face of metal beam guardrail installations.

303.4 Curb Extensions

(1) *Bulbouts*. A bulbout is an extension of the sidewalk into the roadway when there is marked on-street parking, see Index 402.3. <u>Bulbouts should comply with the guidance provided in Figures 303.4A and B; noting that typical features are shown and that the specific site conditions need to be taken into consideration</u>. Bulbouts provide queuing space and shorten crossing distances, thereby reducing pedestrian conflict time with mainline traffic. By placing the pedestrian entry point closer to traffic, bulbouts improve visibility between motorists, bicyclists, and pedestrians. They are most appropriate for urban conventional highways and Rural Main Streets with posted speeds 35 miles per hour or less. Curb extensions are not to extend into Class II Bikeways (Bike Lanes). The corner curb radii should be the minimum needed to accommodate the design vehicle, see Topic 404.

When used, bulbouts should be placed at all corners of an intersection.

When used at mid-block crossing locations, bulbouts should be used on both sides of the street.

The curb face of the bulbout should be setback a minimum of 2 feet as shown in Figures 303.4A and B. See the California MUTCD for on-street parking signs and markings.

Landscaping and appurtenant facilities located within a bulbout are to comply per Topic 405.

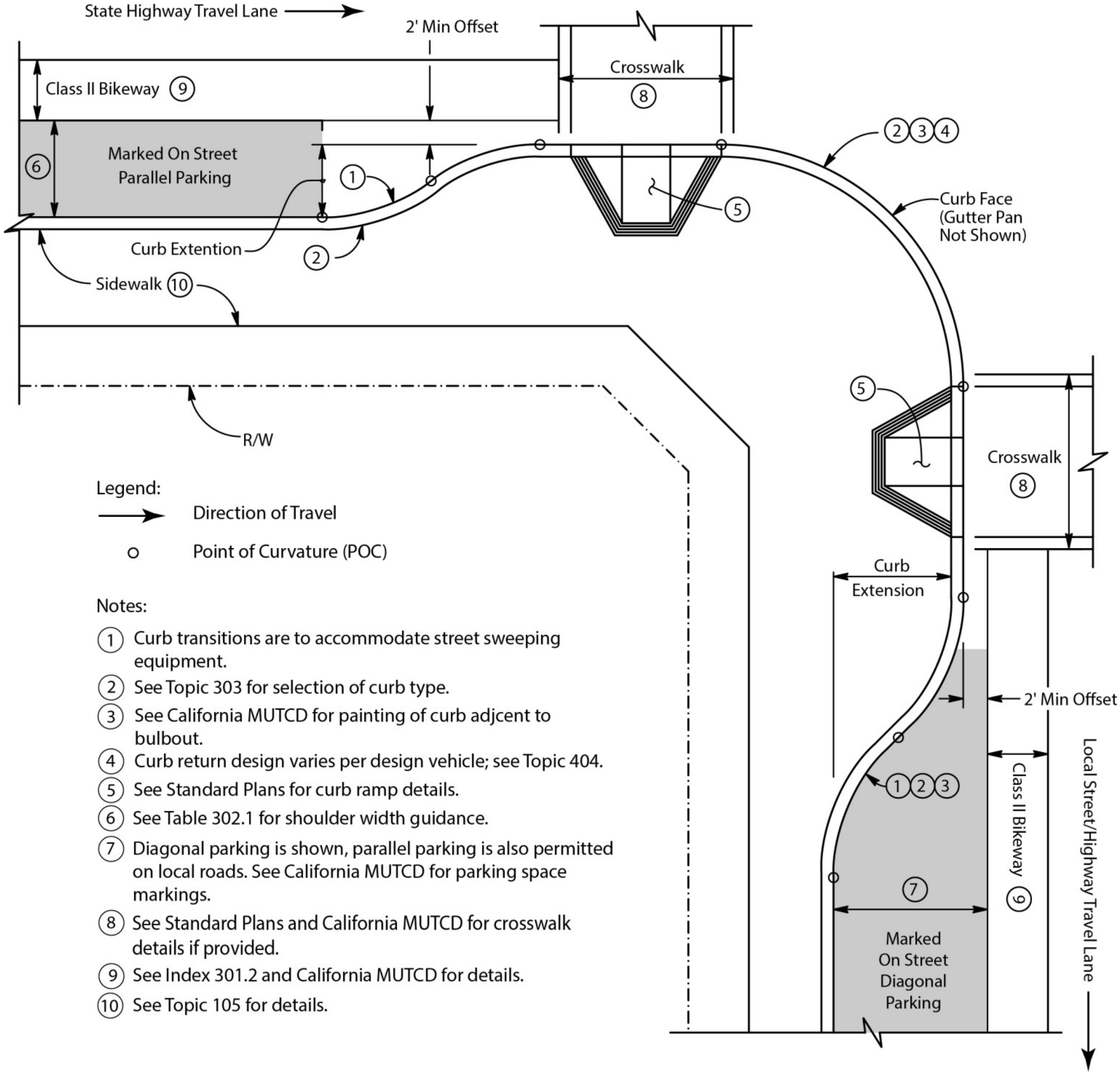
Bulbouts are considered pedestrian facilities and as such, compliance with DIB 82 is required. Avoid bulbouts on facilities where highway grade lines exceed 5 percent.

(2) *Busbulbs.* A busbulb is a bulbout longer than 25 feet which facilitates bus loading and unloading, and provides for enhanced bus mobility. Busbulbs reduce bus dwell times and provide travel time benefits to transit passengers. However, busbulbs can restrict the mobility of vehicular and bicycle traffic because they allow the bus to stop in their traveled way to load and unload passengers. Therefore, their impact on the mobility of the vehicular and bicycle traffic using the facility must be taken into consideration, and pursuant to the California Vehicle Code, busbulbs or other transit stops which require a transit vehicle to stop in the traveled way require approval from the Department. In lieu of a busbulb, a busbay may be considered which will not impact the mobility of the vehicular and bicycle users of the facility.

(3) *Busbays*. A busbay is an indentation in the curb which allows a bus to stop completely outside of vehicular and bicycle lanes.

Figure 303.4A

Typical Bulbout with Class II Bikeway (Bike Lane)



Legend:

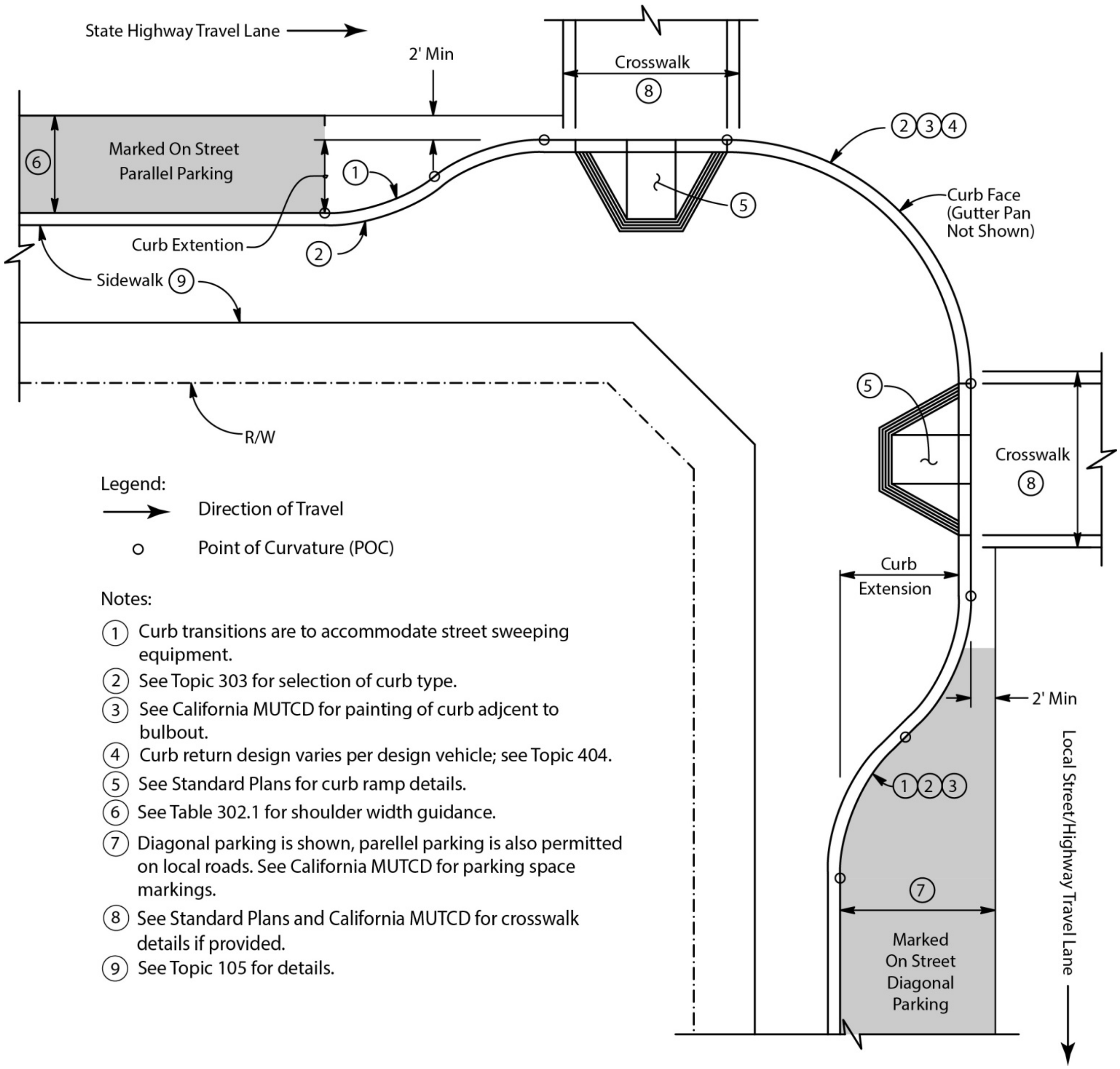
→ Direction of Travel

○ Point of Curvature (POC)

Notes:

1. Curb transitions are to accommodate street sweeping equipment.
2. See Topic 303 for selection of curb type.
3. See California MUTCD for painting of curb adjcent to bulbout.
4. Curb return design varies per design vehicle; see Topic 404.
5. See Standard Plans for curb ramp details.
6. See Table 302.1 for shoulder width guidance.
7. Diagonal parking is shown, parallel parking is also permitted on local roads. See California MUTCD for parking space markings.
8. See Standard Plans and California MUTCD for crosswalk details if provided.
9. See Index 301.2 and California MUTCD for details.
10. See Topic 105 for details.

Figure 303.4B
Typical Bulbout without Class II Bikeway (Bike Lane)



Legend:

→ Direction of Travel

o Point of Curvature (POC)

Notes:

1. Curb transitions are to accommodate street sweeping equipment.
2. See Topic 303 for selection of curb type.
3. See California MUTCD for painting of curb adjcent to bulbout.
4. Curb return design varies per design vehicle; see Topic 404.
5. See Standard Plans for curb ramp details.
6. See Table 302.1 for shoulder width guidance.
7. Diagonal parking is shown, parellel parking is also permitted on local roads. See California MUTCD for parking space markings.
8. See Standard Plans and California MUTCD for crosswalk details if provided.
9. See Topic 105 for details.

Busbays may be created by restricting on street parking.

303.5 Position of Curbs and Dikes

Curbs located at the edge of the traveled way may have some effect on lateral position and speed of moving vehicles, depending on the curb configuration and appearance. Curbs with low, sloped faces may encourage drivers to operate relatively close to them. Curbs with vertical faces may encourage drivers to slow down and/or shy away from them and, therefore, it may be desirable to incorporate some additional roadway width.

All dimensions to curbs (i.e., offsets) are from the near edge of traveled way to bottom face of curb. All dimensions to dikes are from the near edge of traveled way to flow line. Curb and dike offsets should be in accordance with the following:

(1) *Through Lanes.* The offset from the edge of traveled way to the face of curb or dike flow line should be no less than the shoulder width, as set forth in Table 302.1.

(2) *Channelization.* Island curbs used to channelize intersection traffic movements should be positioned as described in Index 405.4.

(3) *Separate Turning Lanes.* Curb offsets to the right of right-turn lanes in urban areas may be reduced to 2 feet if design exception approval for nonstandard shoulder width has been obtained in accordance with Index 82.2. No curb offset is required to the left of left-turn lanes in urban areas unless there is a gutter pan.

(4) *Median Openings.* Median openings (Figure 405.5) should not be separated with curb unless necessary to delineate areas occupied by traffic signal standards.

(5) *Urban Conventional Highways.* When the posted speed is less than or equal to 35 miles per hour, no median curb offset is required if there is no gutter pan.

(6) *Structure Approach Slabs.* When a dike is required to protect the side slope from erosion, it should be placed on the structure approach and sleeper slabs as well as aligned to tie into the end of the structure railing. The guardrail alignment and edge of shoulder govern the positioning of the dike.

When the Type 14 structure approach slab is used, concrete dikes are preferred. Hot mixed asphalt dike will inevitably crack due to expansion and contraction at the approach/sleeper slab joint. A metal dike insert is used to carry the flow across the sealed joint. The insert acts as a water barrier to minimize erosion of the fill slope. Details of the metal dike insert are shown in the structure approach plans provided by the Division of Engineering Services, (DES).

(7) *Bridges and Grade Separation Structures.* When both roadbeds of a curbed divided highway are carried across a single structure, the median curbs on the structure should be in the same location as on adjacent roadways.

(8) *Approach Nose.* The approach nose of islands should also be designed utilizing a parabolic flare, as discussed in Index 405.4.

303.6 Curbs and Dikes on Frontage Roads and Streets

Continuous curbs or dikes are not necessarily required on all frontage roads. Where curbs or dikes are necessary for drainage control or other reasons, they should be consistent with the guidelines established in this topic and placed as shown on Figure 307.4. Local curb standards should be used when requested by local authorities for roads and streets that will be relinquished to them.

Topic 304 - Side Slopes

304.1 Side Slope Standards

Slopes should be designed as flat as is reasonable. <u>For new construction, widening, or where slopes are otherwise being modified, embankment (fill) slopes should be 4:1 or flatter.</u> Factors affecting slope design are as follows:

(a) *Safety.* Flatter slopes provide better recovery for errant vehicles that may run off the road. A cross slope of 6:1 or flatter is suggested for high speed roadways whenever it is achievable. Cross slopes of 10:1 are desirable.

Embankment slopes 4:1 or flatter are recoverable for vehicles. Drivers who encroach on recoverable slopes can generally stop or slow down enough to return to the traveled way safely.

A slope which is between 3:1 and 4:1 is considered traversable, but not recoverable. Since a high percentage of vehicles will reach the toe of these slopes, the recovery area should be extended beyond the toe of slope. The AASHTO Roadside Design Guide should be consulted for methods of determining the preferred extent of the runout area.

Embankment slopes steeper than 3:1 should be avoided when accessible by traffic. District Traffic, and the AASHTO Roadside Design Guide should be consulted for methods of determining the preferred treatment.

Regardless of slope steepness, it is desirable to round the top of slopes so an encroaching user remains in contact with the ground. Likewise, the toe of slopes should be rounded to prevent users from nosing into the ground.

(b) *Erosion Control.* Slope designs steeper than 4:1 must be approved by the District Landscape Architect in order to assure compliance with the regulations affecting Stormwater Pollution contained in the Federal Clean Water Act (see Index 82.4). Slope steepness and length are two of the most important factors affecting the erodibility of a slope. Slopes should be designed as flat as possible to prevent erosion. However, since there are other factors such as soil type, climate, and exposure to the sun, District Landscape Architecture and the District Stormwater Coordinator must be contacted for erosion control requirements.

A Storm Water Data Report (SWDR) documents project information and considerations pertaining to Storm Water Best Management Practices (BMPs) and Erosion Control methods. The SWDR is prepared and signed by key personnel (including the District Landscape Architect) at the completion of each phase of a project. By signing the SWDR, the District Landscape Architect approves compliance with the proposed slope designs.

(c) *Structural Integrity.* Slopes steeper than 2:1 require approval of District Maintenance. The Geotechnical Design Report (See Topic 113) will recommend a minimum slope required to prevent slope failure due to soil cohesiveness, loading, slip planes and other global stability type failures. There are other important issues found in the Geotechnical Design Report affecting slope design such as the consistency of the soil likely to be exposed in cuts, identification of the presence of ground water, and recommendations for rock fall.

(d) *Economics.* Economic factors such as purchasing right of way, imported borrow, and environmental impacts frequently play a role in the decision of slope length and steepness. In some cases, the cost of stabilizing, planting, and maintaining steep slopes may exceed the cost of additional grading and right of way to provide a flatter slope.

(e) *Aesthetics.* Flat, gentle, and smooth, well transitioned slopes are visually more satisfying than steep, obvious cuts and fills. In addition, flatter slopes are more easily revegetated, which helps visually integrate the transportation improvement within its surrounding environment. Contact the District Landscape Architect when preparing a contour grading plan.

<u>In light grading where normal slopes catch in a distance less than 18 feet from the edge of the shoulder, a uniform catch point, at least 18 feet from the edge of the shoulder, should be used</u>. This is done not only to improve errant vehicle recovery and aesthetics, but also to reduce grading costs. Uniform slopes wider than 18 feet can be constructed with large production equipment thereby reducing earthwork costs.

Transition slopes should be provided between adjoining cuts and fills. Such slopes should intersect the ground at the uniform catch point line.

In areas where heavy snowfall can be expected, consideration should be given to snow removal problems and snow storage in slope design. It is considered advisable to use flatter slopes in cuts on the southerly side of the roadway where this will provide additional exposure of the pavement to the sun.

304.2 Clearance From Slope to Right of Way Line

The minimum clearance from the right of way line to catch point of a cut or fill slope should be 10 feet for all types of cross sections. When feasible, at least 15 feet should be provided.

Following are minimum clearances recommended for cuts higher than 30 feet:

(a) Twenty feet for cuts from 30 feet to 50 feet high.

(b) Twenty-five feet for cuts from 50 feet to 75 feet high.

(c) One-third the cut height for cuts above 75 feet, but not to exceed a width of 50 feet.

The foregoing clearance standards should apply to all types of cross sections.

304.3 Slope Benches and Cut Widening

The necessity for benches, their width, and vertical spacing should be finalized only after an adequate materials investigation. Since greater user benefits are realized from widening a cut than from benching the slope, benches above grade should be used only where necessary. Benches above grade should be used for such purposes as installation of horizontal drains, control of surface erosion, or intercepting falling rocks. Design of the bench should be compatible with the geotechnical features of the site.

Benches should be at least 20 feet wide and sloped to form a valley at least 1 foot deep with the low point a minimum of 5 feet from the toe of the upper slope. Access for maintenance equipment should be provided to the lowest bench, and if feasible to all higher benches.

In cuts over 150 feet in height, with slopes steeper than 1½:1, a bench above grade may be desirable to intercept rolling rocks. The Division of Engineering Services – Geotechnical Services (DES-GS) should be consulted for assistance in recommending special designs to contain falling and/or rolling rocks.

Cut widening may be necessary:

(a) To provide for drainage along the toe of the slope.

(b) To intercept and store loose material resulting from slides, rock fall, and erosion.

(c) For snow storage in special cases.

(d) To allow for planting.

Where the widened area is greater than that required for the normal gutter or ditch, it should be flush with the edge of the shoulder and sloped upward or downward on a gentle slope, preferably 20:1 in areas of no snow; and downward on a 10:1 slope in snow areas.

304.4 Contour Grading and Slope Rounding

Contour grading, slope rounding and topsoil replacement are important factors in roadside design to help make highway improvements compatible with the surrounding environment while comply with National Pollutant Discharge Elimination System permits (NPDES). Smooth, flowing contours that tie gracefully into the existing adjacent roadside and landforms are visually appealing and conducive to safe vehicle recovery (see Index 304.1), reduce the potential for erosion and stormwater runoff, and reduce roadside maintenance activities while contributing to the long term success of revegetation planting.

Contour grading plans are to be prepared to facilitate anticipated roadside treatments and future maintenance activities. These plans should show flattened slopes where right of way permits. The tops and ends of all cut slopes should be rounded. Rock cut slopes should be irregular where possible to provide a natural appearance and the tops and ends should also be rounded. All slope designs should include consideration of an application of local or imported topsoil and duff to promote the growth of vegetation, improve stormwater pollutant filtration and control erosion. The calculation of the final grade for a project needs to take into account the reapplication of topsoil and duff.

Local topsoil and duff material within the grading limits should be identified on the plans, removed or excavated, stockpiled, and reapplied. This is to be performed on all projects that include grading or earthwork unless the materials are determined to be unsuitable.

Coordinate the development of contour grading plans including, removal, stockpiling, suitability of material and application of topsoil and duff with the District Landscape Architect.

304.5 Stepped Slopes

Stepped cut slopes should be used to encourage material revegetation from the adjacent plants. Stepped slopes are a series of small benches 1 foot to 2 feet wide. Generally, stepped slopes can be used in rippable material on slopes 2:1 or steeper. Steps may be specified for slopes as flat as 3:1. Steps are provided to capture loose material, seed, 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.

<u>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):</u>

(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, <u>the minimum median width for freeways and</u>

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.

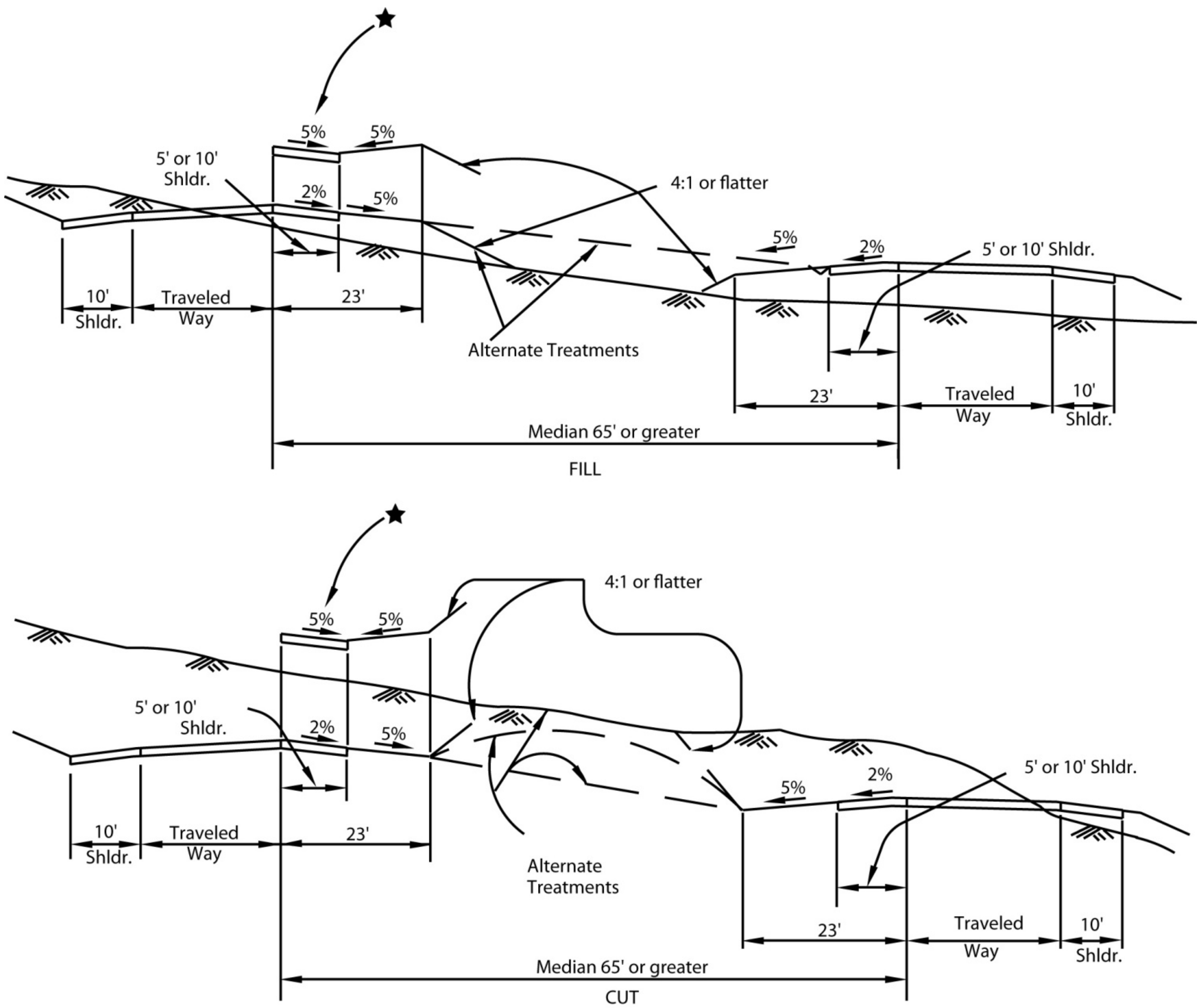
Topic 307 - Cross Sections for State Highways

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.

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

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 width of each shoulder should be 2 feet. Shoulders less than 4 feet are not adequate for bicycles. Where 4-foot shoulders are not possible, consideration should be given to providing turnouts for bicycles. See Index 204.5(4) for turnout information. See Topic 1003 and Index 301.2 for information on bicycle design criteria and Figure 307.2 for typical 2-lane cross sections.

Shoulder widths based on design year traffic volumes shall conform to the standards given in Table 307.2.

Table 307.2

Shoulder Widths for Two-lane Roadbed New Construction Projects

Two-way ADT (Design Year)	Shoulder Width[1] (ft)
Less than 400	4[2]
Over 400	8[3]

NOTES:

(1) See Index 302.1 for shoulder requirements when bike lanes are present.

(2) Minimum bridge width is 32 feet (see Index 208.1).

(3) See Index 405.3(2)(a) for shoulder requirements adjacent to right-turn only lanes.

On 2-lane roads with 4-foot shoulders, the shoulder slope may be increased to 7 percent for additional drainage capacity where a dike is used. A design exception to Index 302.2 will be required to document the decision to increase the slope.

Bicycles are not prohibited on conventional highways: therefore, where the shoulder width is 4 feet, the gutter pan width should be reduced to 1 foot, so 3 feet is provided between the traffic lane and the longitudinal joint at the gutter pan. Whenever possible, grate type inlets should not be located in bicycle paths of travel. See Index 837.2(2) for further grate guidance.

307.3 Two-lane Cross Sections for 2R, 3R, and other Projects

Standards and guidelines for two-lane cross sections on resurfacing and restoration (2R) projects and resurfacing, restoration, and rehabilitation (3R) projects are found in DIB 79 and Index 603.4. DIB 79 also includes screening criteria to determining whether the project fits 2R or 3R.

3R design criteria apply to all structure and roadway 3R projects on two-lane conventional highways and three-lane conventional highways not classified as multilane conventional highways.

3R design criteria also apply to certain storm damage, protective betterment, operational, and safety nonfreeway improvement projects that are considered spot locations as described in detail in DIB 79.

3R criteria apply to geometric design features such as lane and shoulder widths, horizontal and vertical alignment, stopping sight distance, structure width, cross slope, superelevation, side slope, clear recovery zone, curb ramps, pavement edge drop, dike, curb and gutter, and intersections. They may also apply to such features as bike lanes, sidewalk, and drainage.

307.4 Multilane Divided Cross Sections

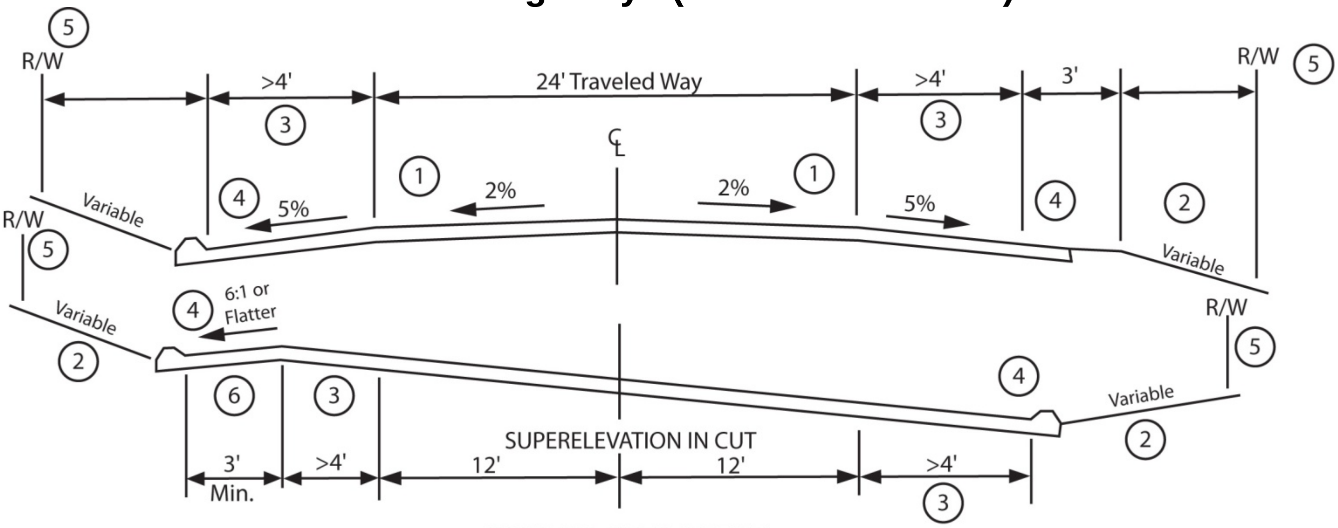
The general geometric features of multilane divided cross sections are shown in Figure 307.4.

Divided highways may be designed as two separate one-way roads where appropriate to fit the terrain. Economy, pleasing appearance, and safety are factors to be considered in this determination. The alignment of each roadway may be independent of the other (see Indexes 204.8 and 305.6). Optional median designs may be as shown on Figure 305.6.

307.5 Multilane All Paved Cross Sections with Special Median Widths

A multilane cross section with a narrow median is illustrated in Figure 307.5. This section is

Figure 307.2
Geometric Cross Sections for Two-lane Highways (New Construction)



STANDARD CROSS SECTION
(Shoulder ≥ 4')

NOTES		
1	CROSS SLOPES	See Index 302.2
2	SIDE SLOPES	See Index 304.1
3	SHOULDER WIDTH	See Index 302.1
4	DIKES	See Index 303.3
5	RIGHT OF WAY	See Index 306.1
6	SIDE GUTTERS	See Index 834.3(3)

Figure 307.4
Geometric Cross Sections for Freeways and Expressways

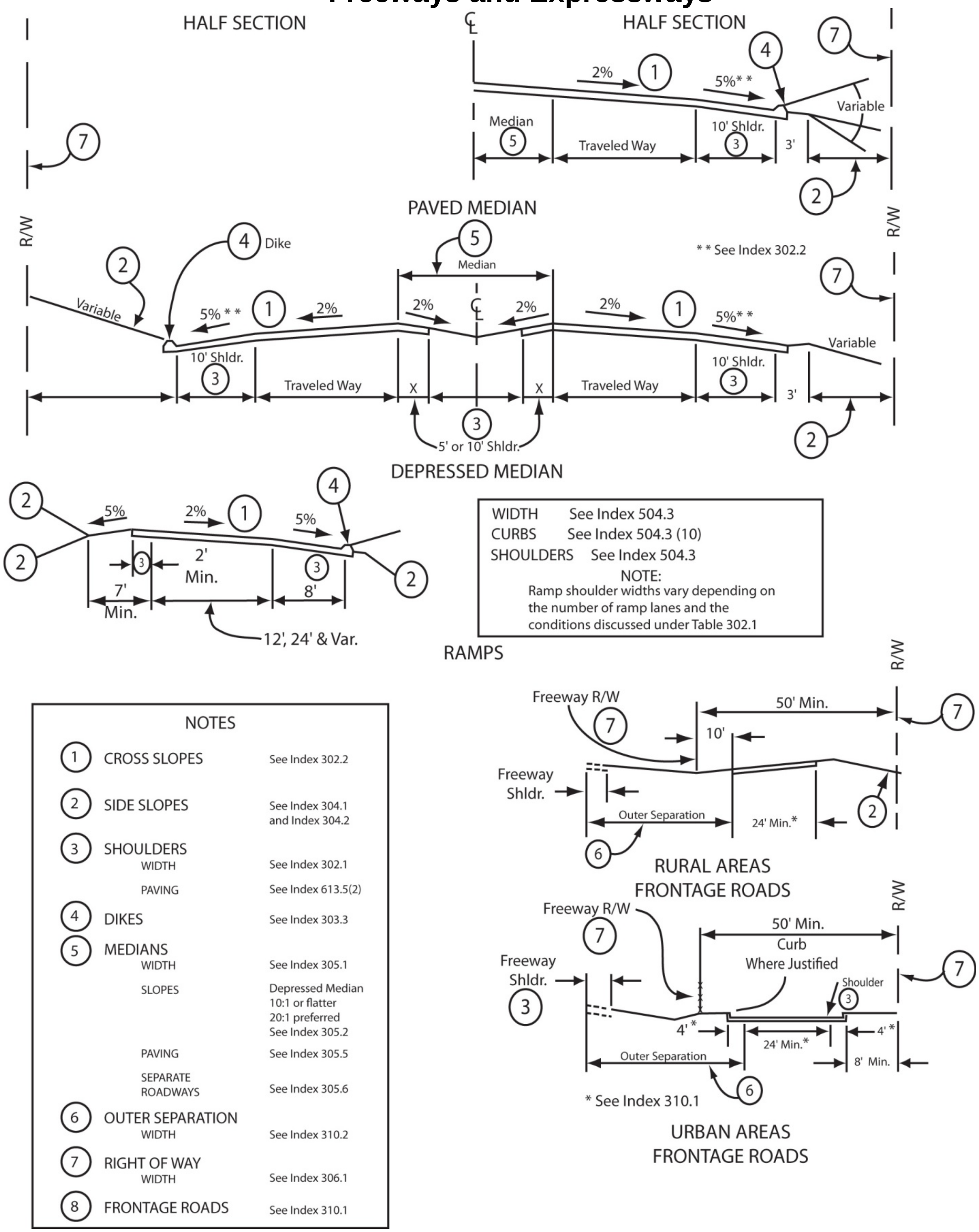
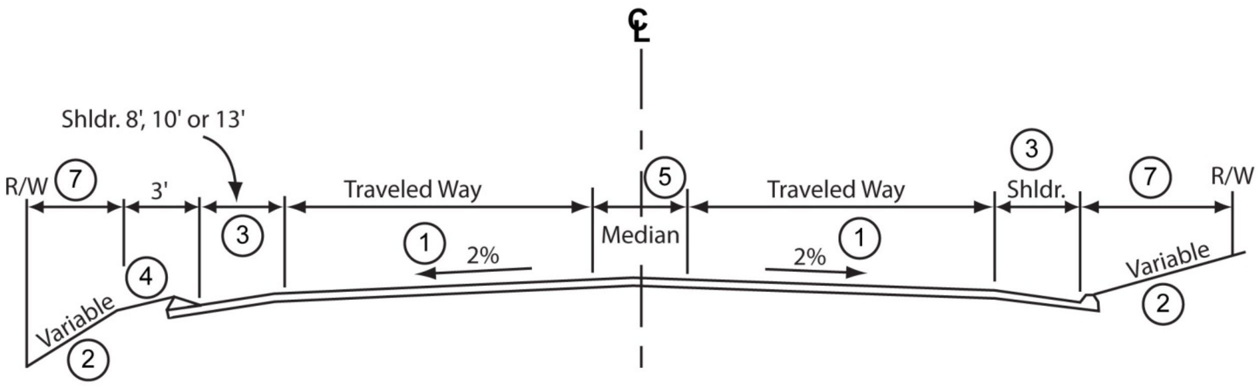
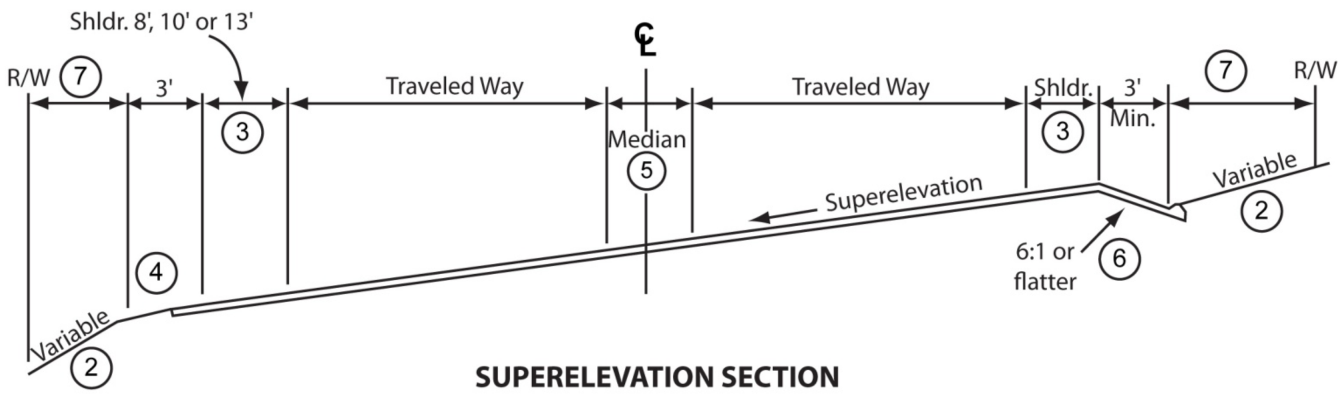


Figure 307.5
Geometric Cross Sections for
All Paved Multilane Highways



TANGENT SECTION



SUPERELEVATION SECTION

NOTES

1	CROSS SLOPES	See Index 302.2
2	SIDE SLOPES	See Index 304.1
3	SHOULDERS	See Index 302.1
4	DIKES	See Index 303.3
5	MEDIANS	See Index 305.1 (3)
6	SIDE GUTTERS	See Index 834.3 (3)
7	RIGHT OF WAY	See Index 306.1

appropriate in special circumstances where a wider median would not be justified. It should not be considered as an alternative to sections with the median widths set forth under Index 305.1. It may be used under the following conditions:

(a) Widening of existing facilities.

(b) Locations where large excavation quantities would result if a multilane roadway cross section with a basic median width were used. Examples are steep mountainous terrain and unstable mountainous areas.

(c) As an alternate cross section on 2-lane roads having frequent sight distance restrictions.

The median width should be selected in accordance with the criteria set forth in Index 305.1(3).

In general, the outside shoulder should be 8 feet wide (10 feet on freeways and expressways) as mandated in Table 302.1. Where large excavation quantities or other factors generate unreasonable costs, 4-foot shoulders may be considered.

However, a design exception is required except where 4-lane passing sections are constructed on 2-lane highways. Where the roadbed width does not contain 8-foot shoulders, emergency parking areas clear of the traveled way should be provided by using daylighted cuts and other widened areas which develop during construction.

307.6 Multilane Cross Sections for 2R and 3R Projects

3R projects on freeways, expressways, and multilane conventional highways are required to meet new construction standards.

For additional information on 2R and 3R projects, see DIB 79.

307.7 Reconstruction Projects

Reconstruction projects on freeways, expressways, and conventional highways are required to meet new construction standards.

Topic 308 - Cross Sections for Roads Under Other Jurisdictions

308.1 City Streets and County Roads

The width of local roads and streets that are to be reconstructed as part of a freeway project should conform to AASHTO standards if the local road or street is a Federal-aid route. Otherwise the cross section should match the width of the city street or county road adjoining the reconstructed portion, or the cross section should satisfy the local agency's minimum standard for new construction.

Where a local facility within the State right of way crosses over or under a freeway or expressway but has no connection to the State facility, the minimum design standards for the cross section of the local facility within the State's right of way shall be those found in AASHTO. <u>If the local agency has standards that exceed AASHTO standards, then the local agency standards should apply.</u>

AASHTO standards for local roads and streets are given in AASHTO, A Policy on Geometric Design of Highways and Streets.

It is important to note that AASHTO, A Policy on Geometric Design of Highways and Streets, standards are based on functional classification and not on a Federal-aid System.

See Chapter 1 of AASHTO, A Policy on Geometric Design of Highways and Streets, for additional information on the AASHTO functional classifications of rural and urban arterials, collector roads, and streets.

AASHTO, A Policy on Geometric Design of Highways and Streets, gives minimum lane and shoulder widths. When selecting a cross section, the effects on capacity of commercial vehicles and grades should be considered as discussed under Topic 102 and in the Transportation Research Board, Highway Capacity Manual.

The minimum width of 2-lane overcrossing structures shall not be less than 32 feet face of curb to face of curb.

If the local agency has definite plans to widen the local street either concurrently or within 5 years following freeway construction, the reconstruction to be accomplished by the State should generally conform to the widening planned by the local agency. Stage construction should be considered where the planned widening will occur beyond the 5-year period following freeway construction or where the local agency has a master plan indicating an ultimate width greater than the existing facility.

Where an undercrossing is involved, the initial structure construction should provide for ultimate requirements.

Where a local facility crosses over or under a freeway or expressway and connects to the State facility (such as ramp terminal intersections), the minimum design standards for the cross section of the local facility shall be at least equal to those for a conventional highway with the exception that the outside shoulder width shall match the approach roadway, but not less than 4 feet, and as shown below.

Where the 2-lane local facility connects to a freeway within an interchange, the lane width of the local facility shall be 12 feet.

Where a multilane local facility connects to a freeway within an interchange, the outer most lane in each direction of the local facility shall be 12 feet.

Shoulder width shall not be less than 5 feet when railings or other lateral obstructions are adjacent to the right edge of shoulder.

If gutter pans are used, then the minimum shoulder width shall be 3 feet wider than the width of the gutter pan being used.

The minimum width for two-lane overcrossing structures at interchanges shall be 40 feet curb-to-curb.

Topic 309 - Clearances

309.1 Horizontal Clearances for Highways

(1) *General.* The horizontal clearance to all roadside objects should be based on engineering judgment with the objective of maximizing the distance between roadside objects and the edge of traveled way. Engineering judgment should be exercised in order to balance the achievement of horizontal clearance objectives and reduction of maintenance cost and exposure to workers, with the prudent expenditure of available funds.

Certain yielding types of fixed objects, such as sand filled barrels, metal beam guardrail, breakaway wood posts, etc. may encroach within the clear recovery zone (see Index 309.1(2)). While these objects are designed to reduce the severity of accidents, efforts should be made to maximize the distance between any object and the edge of traveled way.

Horizontal clearances are measured from the edge of the traveled way to the nearest point on the obstruction (usually the bottom). Consideration should be given to the planned ultimate traveled way width of the highway facility. **Horizontal clearances greater than those cited below under Subsection (3) - "Minimum Clearances" shall be provided where necessary to meet horizontal stopping sight distance requirements.** See subsection (4) for high speed rail clearance guidance. See discussion on "... technical reductions in design speed..." under Topic 101.

(2) *Clear Recovery Zone (CRZ).* The roadside environment can and should be made as safe as practical. A clear recovery zone is an unobstructed, relatively flat (4:1 or flatter) or gently sloping area beyond the edge of the traveled way which affords the drivers of errant vehicles the opportunity to regain control. The AASHTO Roadside Design Guide provides detailed design guidance for creating a forgiving roadside environment. See also Index 304.1 regarding side slopes.

The following clear recovery zone widths are the minimum desirable for the type of facility indicated. Consideration should be given to increasing these widths based on traffic volumes, operating speeds, terrain, and costs associated with a particular highway facility:

- Freeways and Expressways – 30 feet
- Conventional Highways – 20 feet*

* On conventional highways with posted speeds less than or equal to 35 miles per hour and curbs, clear recovery zone widths do not apply. See minimum horizontal clearance, Index 309.1(3)(c).

(a) Necessary Highway Features.

Fixed objects, when they are necessary highway features, including, but not limited to, bridge piers, abutments, retaining walls, and noise barriers closer to the edge of traveled way than the distances listed above should be eliminated, moved, redesigned to be made yielding, or shielded in accordance with the following guidelines:

- Fixed objects, when they are necessary highway features, should be eliminated or moved outside the clear recovery zone to a location where they are unlikely to be hit.
- If necessary highway features such as sign posts or light standards cannot be eliminated or moved outside the clear recovery zone, they should be made yielding with a breakaway feature.
- If a fixed object, when they are necessary highway features, cannot be eliminated, moved outside the clear recovery zone, or modified to be made yielding, it should be shielded by guardrail, barrier or a crash cushion.

Shielding and breakaway features must be in conformance with the guidance found in Chapter 7 of the Traffic Manual. For input on the need for shielding at a specific location, consult District Traffic Operations.

When the planting of trees is being considered, see the additional discussion and standards in Chapter 900.

(b) Discretionary Fixed Objects.

Discretionary fixed objects are features or facilities that are not necessary for the safety, maintenance or operation of the highway, but may enhance livability and sustainability. These may include, but are not limited to, transportation art, gateway monuments, solar panels, and memorial/historical plaques or markers. See Subsection (4) for high speed rail clearance guidance. When discretionary fixed objects are constructed on freeways, expressways or conventional highways without curbs and posted speeds over 35 mph, they should be located beyond the clear recovery zone, at a minimum of 52 feet horizontally or 8 feet vertically up-slope from the planned ultimate edge of traveled way. If discretionary fixed objects are to be placed less than the 52 feet horizontally or less than the 8 feet vertically up-slope, they should be made breakaway or shielded behind existing guardrail, barrier or other safety device.

Where compliance with the guidelines stated in Subsections (2)(a) and (b) are impractical, the minimum horizontal clearance cited in Subsection (3) Minimum Clearances shall apply to the unshielded fixed object. These minimum horizontal clearances apply to yielding objects as well.

(3) *Minimum Clearances.* **The following minimum horizontal clearances shall apply to all objects that are closer to the edge of traveled way than the clear recovery zone distances listed above:**

(a) **The minimum horizontal clearance to all objects, such as bridge rails and safety-shaped concrete barriers, as well as sand-filled barrels, metal beam guardrail, etc., on all freeway and expressway facilities, including auxiliary lanes, ramps, and collector-distributor roads, shall be equal to the standard shoulder width of the highway facility as stated in Table 302.1. A minimum clearance of 4 feet shall be provided where the standard shoulder width is less than 4 feet. Approach rail connections to bridge rail may require special treatment to maintain the standard shoulder width.**

(b) **The minimum horizontal clearance to walls, such as abutment walls, retaining walls in cut locations, and noise barriers on all facilities, including auxiliary lanes, ramps and collector-distributor roads, shall not be less than 10 feet per Table 302.1.**

(c) **On conventional highways, frontage roads, city streets and county roads within the State right of way (all without curbs), the minimum horizontal**

clearance shall be the standard shoulder width as listed in Tables 302.1 and 307.2, except that a minimum clearance of 4 feet shall be provided where the standard shoulder width is less than 4 feet. For RRR projects, widths are provided in DIB 79.

On conventional highways with curbs, typically in urban conditions, a minimum horizontal clearance of 1 foot 6 inches should be provided beyond the face of curbs to any obstruction. On curbed highway sections, a minimum clearance of 3 feet should be provided along the curb returns of intersections and near the edges of driveways to allow for design vehicle offtracking (see Topic 404). Where sidewalks are located immediately adjacent to curbs, fixed objects should be located beyond the back of sidewalk to provide an unobstructed area for pedestrians.

In areas without curbs, the face of Type 60 concrete barrier should be constructed integrally at the base of any retaining, pier, or abutment wall which faces traffic and is 15 feet or less from the edge of traveled way (right or left of traffic and measured from the face of wall). See Index 1102.2 for the treatment of noise barriers.

The minimum width of roadway openings between Temporary Railing (Type K) on bridge deck widening projects should be obtained from the HQ Transportation Permit Program.

The HQ Transportation Permit Program must be consulted on the use of the route by overwidth loads.

See Chapter 7 of the Traffic Manual for other requirements pertaining to clear recovery zone, guardrail at fixed objects and embankments, and crash cushions.

(4) *High Speed Rail Clearances.* When a high speed rail corridor is to be constructed longitudinally to a freeway, expressway or a conventional highway with posted speeds over 40 miles per hour, the nearest fixed object or feature associated with the operation of the rail facility should be located a minimum of 52 feet horizontally from the planned ultimate edge of the traveled way. See Index 62.10 for the definition of high speed rail. The terrain and the required highway features between the edge of traveled way and the rail facility to be constructed must be evaluated to determine on a case-by-case basis whether or not shielding behind guardrail, barrier or other safety device in conformance with the guidance found in Chapter 7 of the Traffic Manual is needed. For input on the need for shielding at a specific location, consult District Traffic Operations.

(5) *Other Transportation Facilities.* Contraflow BRT, light rail facilities, and heavy rail facilities are considered fixed objects and the clearances noted in Index 309.1 apply.

Parallel BRT facilities are preferred to have the following minimum separation between lanes:

- Freeways and Expressways** – 4 feet
- Conventional Highways (see also Index 108.5)
  - Posted Speeds over 40 miles per hour – 4 feet
  - Posted Speeds equal or greater than 25 miles per hour and up to 45 miles per hour in an urban environment – 2 feet, with curbed separation, 4 feet with 2-foot curbed separation recommended.

** See "A Guide for HOT Lane Development", FHWA, and Caltrans High Occupancy Vehicle Guidelines for additional information.

309.2 Vertical Clearances

(1) *Major Structures.*

(a) Freeways and Expressways, All construction except overlay projects – **16 feet 6 inches shall be the minimum vertical clearance over the roadbed of the State facility (e.g., main lanes, shoulders, ramps, collector-distributor roads, speed change lanes, etc.).**

Table 309.2A

Minimum Vertical Clearances

	Traveled Way	Shoulder
Freeways and Expressways, New Construction, Lane Additions, Reconstruction and Modification	16½ ft	16½ ft
Freeways and Expressways, Overlay Projects	16 ft	16 ft
All Projects on Conventional Highways and Local Facilities	15 ft	14½ ft
Sign Structures	18 ft	18 ft
Pedestrian, Bicycle Overcrossings, and Minor Structures	Standard + 2 ft See 309.2(2)	
Structures on the Rural and Single Interstate Routing System	See 309.2(3)	

Figure 309.2
Department of Defense
Rural and Single Interstate Routes

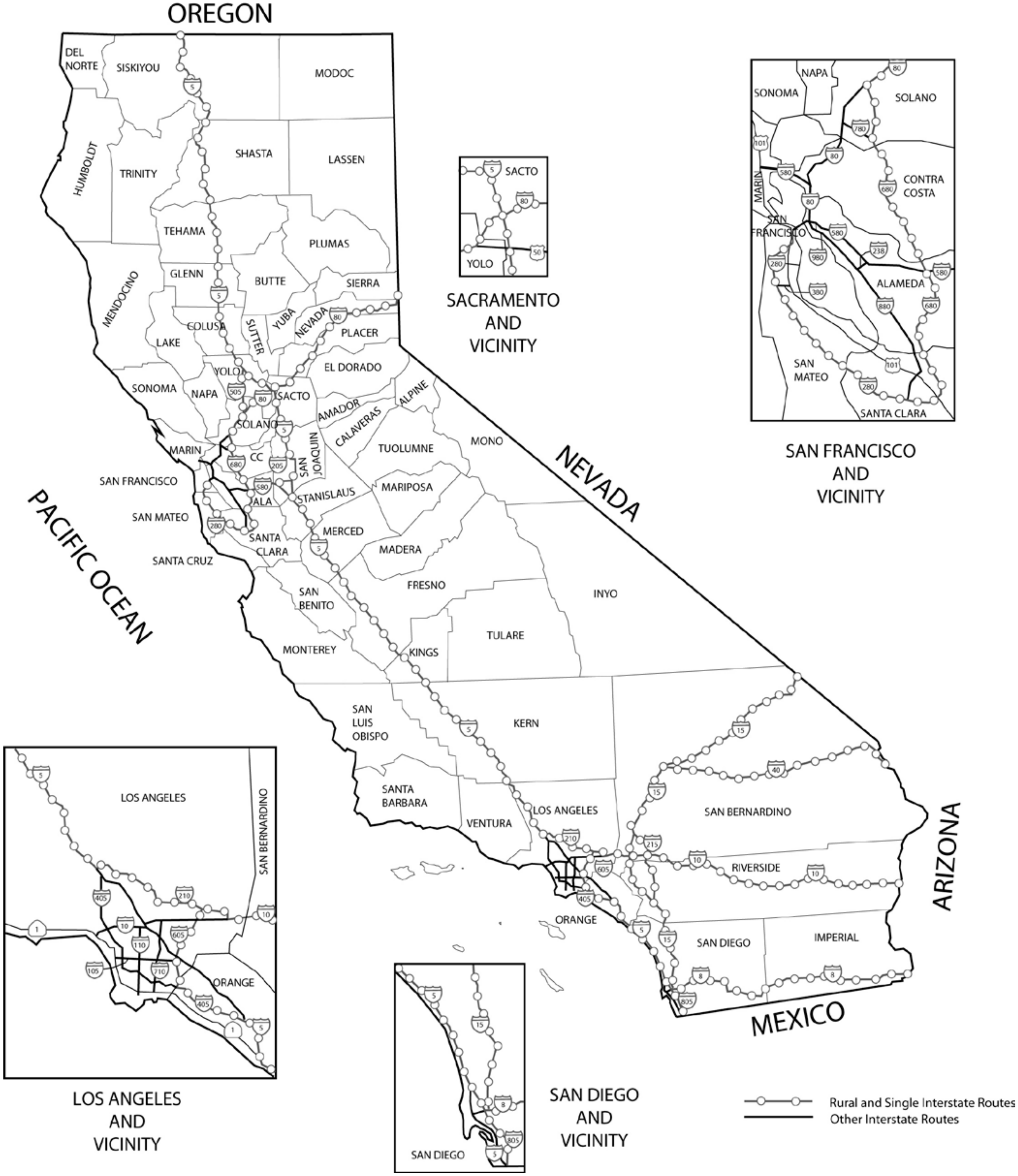


Table 309.2B
California Routes on the Rural and Single Interstate Routing System

ROUTE	FROM	TO
I-5	U. S. Border	I-805 just N. of U. S. Border
I-5	I-805 N. of San Diego	I-405 near El Toro
I-5	I-210 N. of Los Angeles	Oregon State Line
I-8	I-805 near San Diego	Arizona State Line
I-10	I-210 near Pomona	Arizona State Line
I-15	I-8 near San Diego	Nevada State Line
I-40	Junction at I-15 near Barstow	Arizona State Line
I-80	I-680 near Cordelia	Nevada State Line
I-205	Junction at I-580	Junction at I-5
I-210	I-5 N. of Los Angeles	I-10 near Pomona
I-215	I-15 near Temecula	I-15 near Devore
I-280	Junction at I-680 in San Jose	At or near south city limits of San Francisco to provide access to Hunter's Point
I-405	I-5 near El Toro	Palo Verde Avenue just N. of I-605
I-505	Junction at I-80	Junction at I-5
I-580	I-680 near Dublin	Junction at I-5
I-605	I-405 near Seal Beach	I-210
I-680	Junction at I-280 in San Jose	I-80 near Cordelia
I-805	I-5 just N. of U. S. Border	I-5 N. of San Diego

(b) Freeways and Expressways, Overlay Projects – **16 feet shall be the minimum vertical clearance over the roadbed of the State facility.**

(c) Conventional Highways, Parkways, and Local Facilities, All Projects – **15 feet shall be the minimum vertical clearance over the traveled way and 14 feet 6 inches shall be the minimum vertical clearance over the shoulders of all portions of the roadbed.**

(2) *Minor Structures.* **Pedestrian over-crossings shall have a minimum vertical clearance 2 feet greater than the standard for major structures for the State facility in question.**

Sign structures shall have a vertical clearance of 18 feet over the roadbed of the State facility.

(3) *Rural Interstates and Single Routing in Urban Areas:* This subset of the Interstate System is composed of all rural Interstates and a single routing in urban areas. Those routes described in Table 309.2B and Figure 309.2 are given special attention in regards to minimum vertical clearance as a result of agreements between the FHWA and the Department of Defense. **Vertical clearance for structures on this system shall meet the standards listed above for freeways and expressways.** In addition to the standards listed above, vertical clearances of less than 16 feet over any portion of this system must be approved by FHWA in coordination with Surface Deployment and Distribution Command Transportation Engineering Agency (SDDCTEA). Documentation in the form of a Design Exception Fact Sheet must be submitted to FHWA to obtain approval for less than 16 feet of vertical clearance. Vertical clearances of less than 16 feet over any Interstate will require FHWA/SDDCTEA notification. See http://www.fhwa.dot.gov/design/090415.cfm

(4) *General Information.* The standards listed above and summarized in Table 309.2A are the minimum allowable on the State highway system for the facility and project type listed. For the purposes of these vertical clearance standards, all projects on the freeway and expressway system other than overlay projects shall be considered to be covered by the "new construction" standard.

When approved by a design exception (see HDM Index 82.2) clearances less than the values given above may be allowed on a case by case basis given adequate justification based upon engineering judgment, economic, environmental or right of way considerations. Typical instances where lesser values may be approved are where the structure is protected by existing lower structures on either side or where a project includes an existing structure that would not be feasible to modify to the current standard. In no case should vertical clearance be reduced below 15 feet over the traveled way or 14 feet 6 inches over the shoulders over any portion of a State highway facility.

Efforts should be made to avoid decreasing the existing vertical clearance whenever possible and consideration should be given to the feasibility of increasing vertical clearance on projects involving structural section removal and replacement. Any project that would reduce vertical clearances below 16 feet 6 inches or lead to an increase in the vertical clearance should be brought to the attention of the Project Delivery Coordinator or District approval authority, depending upon the current District Design Delegation Agreement, the District Permit Engineer and the Regional Permit Manager at the earliest possible date.

The Regional Permit Manager should be informed of any changes (temporary or permanent) in vertical clearance.

(5) *Federal Aid Participation.* Federal-aid participation is normally limited to the following maximum vertical clearances unless there are external controls such as the need to provide for falsework clearance or the vertical clearance is controlled by an adjacent structure in a multi-structure interchange:

(a) Highway Facilities.

- 17 feet over freeways and expressways.

- 15 feet 6 inches over other highways (15 feet over shoulders).
- For pedestrian structures, 2 feet greater than the above values.

(b) Railroad Facilities.

- 23 feet 4 inches over the top of rails for non-electrified rail systems.
- 24 feet 3 inches over the top of rails for existing or proposed 25 kv electrification.
- 26 feet over the top of rails for existing or proposed 50 kv electrification.

These clearances include an allowance for future ballasting of the rail facility. The cost of reconstructing or modifying any existing railroad-highway grade separation structure solely to accommodate electrification will not be eligible for Federal-aid highway fund participation. Where a rail system is not currently electrified, the railroad must have a plan adopted which specifies the intent to electrify the subject rail segment within a reasonable time frame in order to provide clearances in excess of 23 feet 4 inches.

Any exceptions to the clearances listed above should be reviewed with the FHWA early in the design phase to ensure that they will participate in the structure costs. All excessive clearances should be documented in the project files. Documentation must include reasons for exception including the railroad's justification for increased vertical clearance based on an analysis of engineering, operational and/or economic conditions at a specific structure location with appropriate approval by the HQ Right of Way, Railroad Agreement Coordinator and concurrence by the FHWA.

See Index 1003.1(2) for guidance on Class I bikeway vertical clearance.

309.3 Tunnel Clearances

(1) *Horizontal Clearances.* Tunnel construction is so infrequent and costly that the width should be considered on an individual basis. For the minimum width standards for freeway tunnels see Index 309.1.

Normally, the minimum horizontal clearance on freeways should include the full roadbed width of the approaches.

In one-way tunnels on conventional highways the minimum side clearance from the edge of the traveled way shall be 4 feet 6 inches on the left and 6 feet on the right. For two-way tunnels, this clearance shall be 6 feet on each side. This clearance provides space for bicycle lanes or for bicyclists who want to use the shoulder.

(2) *Vertical Clearances.* **The minimum vertical clearance shall be 15 feet measured at any point over the traveled way and 14 feet 6 inches above the gutter at the curb line. On freeways and expressways, the vertical clearance listed in Index 309.2(1)(a) shall be used.** Cost weighed against the probability of over-height vehicles will be the determining factors.

309.4 Lateral Clearance for Elevated Structures

Adequate clearance must be provided for maintenance, repair, construction, or reconstruction of adjacent buildings and of the structure; to avoid damage to the structure from a building fire or to buildings from a vehicle fire; to permit operation of equipment for fire fighting and other emergency teams. **The minimum horizontal clearance between elevated highway structures, such as freeway viaducts and ramps, and adjoining buildings or other structures shall be 15 feet for single-deck structures and 20 feet for double-deck structures. Spot encroachments on this clearance shall be approved in accordance with Index 82.2.**

309.5 Structures Across or Adjacent to Railroads

Regulations governing clearances on railroads and street railroads with reference to side and overhead structures, parallel tracks, crossings of public roads, highways, and streets are established by the PUC. The PUC requirements are minimums for all grade separated structures. The railroad clearances are much greater due to operational requirements.

(1) *Normal Horizontal and Vertical Clearances.* Although General Order No. 26-D specifies a minimum vertical clearance of 22 feet 6 inches above tracks on which freight cars not exceeding a height of 15 feet 6 inches are transported, a minimum of 23 feet 4 inches should be used in design to allow for reballasting and normal maintenance of track. Railroads on which freight cars are not operated, should have a minimum vertical clearance of 19 feet. See Index 309.2(5)(b) for FHWA maximums. In establishing the grade line, the District should consult the DES to obtain the depth of structures and false work requirements, if any (see Index 204.6(4)).

Horizontal clearance from piers, abutments, and barriers shall be 25 feet minimum to centerline of track. For clearances less than 25 feet, the piers supporting bridges over the railroads are to be heavy construction or are to be protected by a reinforced concrete crash wall. Piers are to be considered heavy construction if they have a cross-sectional area equal to or greater than that required for the crash wall where the larger of its dimension is parallel to the track.

Crash walls for piers from 12 to 25 feet clearance from the centerline of track are to have a minimum height of 6 feet above the top of rail. Piers less than 12 feet clearance from the centerline of track are to have a minimum crash wall height of 12 feet above the top of rail. Horizontal clearances other than those stated above must be approved by the PUC and concurred by the affected railroad entity. Coordinate early in the design phase of the project with the District Railroad Coordinator when railroad agreements are required.

For future planned track expansion, a minimum horizontal clearance distance of 20 feet between existing and future track centerlines shall be provided for freight tracks and 25 feet for commuter tracks. See Figure 309.5A for typical horizontal railroad clearances and Figure 309.5B for limits of permanent vertical clearance envelope for grade separated structures.

Code of Federal Regulations 646.212(a)(2) provides that if the railroad establishes to the satisfaction of the Department and FHWA that it has definite demand and plans for installation of additional tracks within a reasonable time, for grade separation structures, Federal funds may be used to provide space for more tracks than are in place.

Vertical clearance greater than 23 feet 4 inches may be approved on a site by site basis where justified by the railroad to the satisfaction of the Department and the FHWA. A railroad's justification for increased vertical clearance should be based on an analysis of engineering, operational and/or economic conditions and the need for future tracks at a specific location. Contact the District Railroad Coordinator for further information.

Table 309.5A
Minimum Vertical Clearances Above Highest Rail

Type of Structure	Type of Operation	
	Normal Freight	No Freight Cars Operated
Highway overhead and other structures including through railroad bridges.	23' – 4"	19' – 0"

At underpasses, General Order No. 26-D establishes a minimum vertical clearance of 15 feet above any public road, highway or street. **However, the greater clearances specified under Index 309.2 shall be used.**

For at grade crossings, all curbs, including median curbs, should be designed with 10 feet of clearance from the track centerline measured normal thereto.

(2) *Off-track Maintenance Clearance.* The 18-foot horizontal clearance is intended for sections of railroad where the railroad company is using or definitely plans to use off-track maintenance equipment. This

clearance is provided on one side of the railroad right of way.

On Federal-aid projects, where site conditions are such that off-track maintenance clearance at an overhead is obtained at additional cost, Federal-aid funds may participate in the costs of such overhead designs that provide up to 18 feet 2 inches horizontal clearance on one side of the track. In such cases, the railroad is required to present a statement that off-track maintenance equipment is being used, or is definitely planned to be used, along that section of the railroad right of way crossed by the overhead structure.

(3) *Walkway Clearances Adjacent to Railroads At Grade.* All plans involving construction adjacent to railroads at grade should be such that there is no encroachment on the walkway adjoining the track. Walkway requirements are set forth in General Order No. 118 of the PUC. Where excavations encroach into walkway areas, the contractor is required to construct a temporary walkway with handrail as set forth in the contract special provisions.

(4) *Approval.* All plans involving clearances from a railroad track must be submitted to the railroad for approval as to railroad interests. Such clearances are also subject to approval by the PUC.

To avoid delays, early consideration must be given to railroad requirements when the planning phase is started on a project.

Topic 310 - Frontage Roads

310.1 Cross Section

Frontage roads are normally relinquished to local agencies. When Caltrans and a county or city enter into an agreement (cooperative agreement, freeway agreement, or other type of binding agreement), the CTC may relinquish to the county or city any frontage or service road or outer highway within that city or county. The relinquished right of way (called a collateral facility) should be at least 40 feet wide and have been constructed as part of a State highway project. Index 308.1 gives width criteria for city streets and county roads. These widths are also applicable to frontage roads. **However, the minimum paved 2-lane cross section width including 4-foot shoulders without curb and gutter shall be:**

- **32 feet if 12-foot lanes are to be provided;**
- **30 feet if 11-foot lanes are to be provided.**

The minimum paved 2-lane cross section width, including 5-foot shoulders and curb and gutter shall be:

- **34 feet if 12-foot lanes are to be provided;**
- **32 feet if 11-foot lanes are to be provided.**

310.2 Outer Separation

<u>In urban areas and in mountainous terrain, the width of the outer separation should be a minimum of 26 feet from edge of traveled way to edge of traveled way.</u> A greater width may be used where it is obtainable at reasonable additional cost, for example, on an urban highway centered on a city block and paralleling the street grid.

<u>In rural areas, other than mountainous terrain, the outer separation should be a minimum of 40 feet wide from edge of traveled way to edge of traveled way.</u>

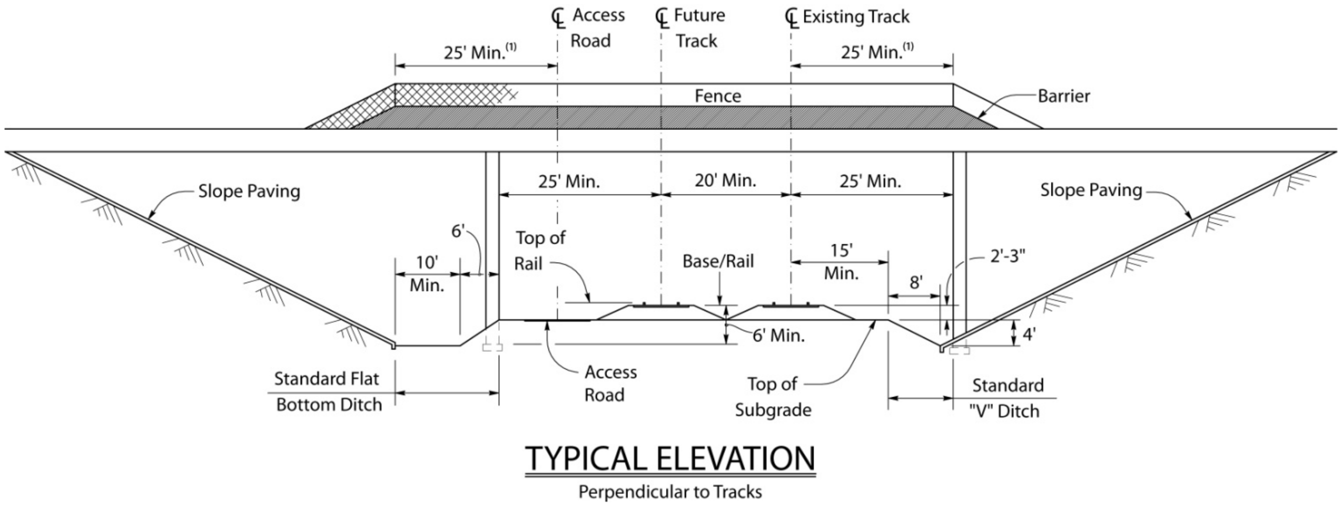
See Figure 307.4 for cross sections of outer separation and frontage road.

310.3 Headlight Glare

Care should be taken when designing new frontage roads to avoid the potential for headlight glare interfering with the vision of motorists, bicyclists, and pedestrians traveling in opposite directions on the frontage roads and in the outer freeway lanes. Consideration should also be given to bike and pedestrians paths. To prevent headlight glare interference on new construction, the preferred measures are for wider outer separations, revised alignment and raised or lowered profiles.

Figure 309.5A

Typical Horizontal Railroad Clearance from Grade Separated Structures

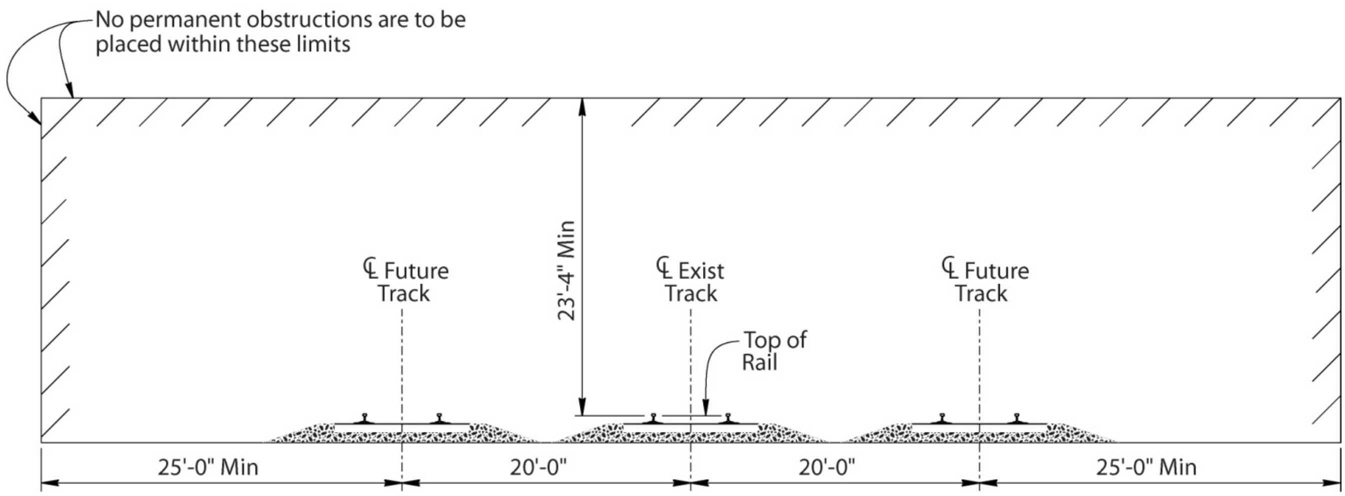


NOTE:

The limits of the fence with barrier rail should extend to the limits of railroad right-of-way or a minimum of 25 feet beyond the centerline of the outermost existing track, future track or access roadway, whichever is greater.

Figure 309.5B

Permanent Railroad Clearance Envelope



(NORMAL TO RAILROAD)

No Scale

Table 309.5B

Minimum Horizontal Clearances to Centerline of Nearest Track

Type of Structure	Off-track Maintenance Clearance	Tangent Track Clearance	Normal Curved Track (1) Clearance	Curved Track Clearances When Space is Limited(1): Curves of 0° to 12°	Curved Track Clearances When Space is Limited(1): Curves of 12° or more
Through railroad bridge	None	8' – 0"(2)(4)	9' – 0"(2)(4)		
Highway overhead and other structures	18' – 0" clear to face of pier or abutment on side railroad requires for equipment road.	8' – 6"(4)	9' – 6"(4)	8' – 6" (Min.)(3)	8' – 6" + ½"(3) per degree of curve.
Curbs		10' – 0"			

NOTES:

(1) The minimum, in general, is one foot greater than for tangent track.

(2) With approval of P.U.C.

(3) Greater clearance necessary if walkway is required.

(4) Collision walls may be required. See Index 309.5(1).

CHAPTER 400
INTERSECTIONS AT GRADE

Intersections are planned points of conflict where two or more roadways join or cross. At-grade intersections are among the most complicated elements on the highway system, and control the efficiency, capacity, and safety for motorized and non-motorized users of the facility. The type and operation of an intersection is important to the adjacent property owners, motorists, bicyclists, pedestrians, transit operators, the trucking industry, and the local community.

There are two basic types of at grade intersections: crossing and circular. It is not recommended that intersections have more than four legs. Occasionally, local development and land uses create the need for a more complex intersection design. Such intersections may require a specialized intersection design to handle the specify traffic demands at that location. In addition to the guidance in this manual, see Traffic Operations Policy Directive (TOPD) Number 13-02: Intersection Control Evaluation (ICE) for direction and procedures on the evaluation, comparison and selection of the intersection types and control strategies identified in Index 401.5. Also refer to the Complete Streets Intersection Guide for further information.

Topic 401 - Factors Affecting Design

Index 401.1 - General

At-grade intersections must handle a variety of conflicts among users, which includes truck, transit, pedestrians, and bicycles. These recurring conflicts play a major role in the preparation of design standards and guidelines. Arriving, departing, merging, turning, and crossing paths of moving pedestrians, bicycles, truck, and vehicular traffic have to be accommodated within a relatively small area. The objective of designing an intersection is to effectively balance the convenience, ease, and comfort of the users, as well as the human factors, with moving traffic (automobiles, trucks, motorcycles, transit vehicles, bicycles, pedestrians, etc.).The safety and mobility needs of motorist, bicyclist and pedestrians as well as their movement patterns in intersections must be analyzed early in the planning phase and then followed through appropriately during the design phase of all intersections on the State highway. It is Departmental policy to develop integrated multimodal projects in balance with community goals, plans, and values.

The Complete Intersections: A Guide to Reconstructing Intersections and Interchanges for Bicyclists and Pedestrians contains a primer on the factors to consider when designing intersections. It is published by the California Division of Traffic Operations.

401.2 Human Factors

(1) *The Driver.* An appreciation of driver performance is essential to proper highway design and operation. The suitability of a design rests as much on how safely and efficiently drivers are able to use the highway as on any other criterion.

Motorist's perception and reaction time set the standards for sight distance and length of transitions. The driver's ability to understand and interpret the movements and crossing times of the other vehicle drivers, bicyclists, and pedestrians using the intersection is equally important when making decisions and their associated reactions. The designer needs to keep in mind the user's limitations and therefore design intersections so that they meet user expectation.

(2) *The Bicyclist.* Bicyclist experience, skills and physical capabilities are factors in intersection design. Intersections are to be designed to help bicyclists understand how to traverse the intersection. Chapter 1000 provides intersection guidance for Class I and Class III bikeways that intersect the State highway system. The guidance in this chapter specifically relates to bicyclists that operate within intersections on the State highway system.

(3) *The Pedestrian.* Understanding how pedestrians will use an intersection is critical because pedestrian volumes, their age ranges, physical ability, etc. all factor in to their startup time and the time it takes them to cross an intersection and thus, dictates how to design

the intersection to avoid potential conflicts with bicyclists and motor vehicles. The guidance in this chapter specifically relates to pedestrian travel within intersections on the State highway system. See Topic 105, Pedestrian Facilities, Design Information Bulletin 82 - "Pedestrian Accessibility Guidelines for Highway Projects," the AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities, and the California Manual on Uniform Traffic Control Devices (California MUTCD) for additional guidance.

401.3 Traffic Considerations

Good intersection design clearly indicates to bicyclists and motorists how to traverse the intersection (see Figure 403.6A). Designs that encourage merging traffic to yield to through bicycle and motor vehicle traffic are desirable.

The size, maneuverability, and other characteristics of bicycles and motorized vehicles (automobiles, trucks, transit vehicles, farm equipment, etc.) are all factors that influence the design of an intersection. The differences in operating characteristics between bicycles and motor vehicles should be considered early in design.

Table 401.3 compares vehicle characteristics to intersection design elements.

A design vehicle is a convenient means of representing a particular segment of the vehicle population. See Topic 404 for a further discussion of the uses of design vehicles.

Transit vehicles and how their stops interrelate with an intersection, pedestrian desired walking patterns and potential transfers to other transit facilities are another critical factor to understand when designing an intersection. Transit stops and their placement needs to take into account the required maintenance operations that will be needed and usually supplied by the Transit Operator.

401.4 The Physical Environment

In highly developed urban areas, where right of way is usually limited, the volume of vehicular traffic, pedestrians, and bicyclists may be large, street parking exists, and transit stops (for both buses and light rail) are available. All interact in a variety of movements that contribute to and add to the complexity of a State highway and can result in busy intersections.

Industrial development may require special attention to the movement of large trucks.

Rural areas where farming occurs may require special attention for specialized farm equipment. In addition, rural cities or town centers (rural main streets) also require special attention.

Rural intersections in farm areas with low traffic volumes may have special visibility problems or require shadowing of left-turn vehicles from high speed approach traffic.

Table 401.3

Vehicle Characteristics	Intersection Design Element Affected
Length	Length of storage lane
Width	Lane width
Height	Clearance to overhead signs and signals
Wheel base	Corner radius and width of turning lanes
Acceleration	Tapers and length of acceleration lane
Deceleration	Tapers and length of deceleration lane

There are many factors to be considered in the design of intersections, with the goal to achieve a functional, safe and efficient intersection for all users of the facility. The location and level of use by various modes will have an impact on intersection design, and therefore should be considered early in the design process. In addition to current levels of use, it is important to consider future travel patterns for vehicles, including trucks; pedestrian and bicycle demand and the future expansion of transit.

401.5 Intersection Type

Intersection types are characterized by their basic geometric configuration, and the form of intersection traffic control that is employed:

(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. <u>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.</u> (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 District Design Liaison 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.

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)

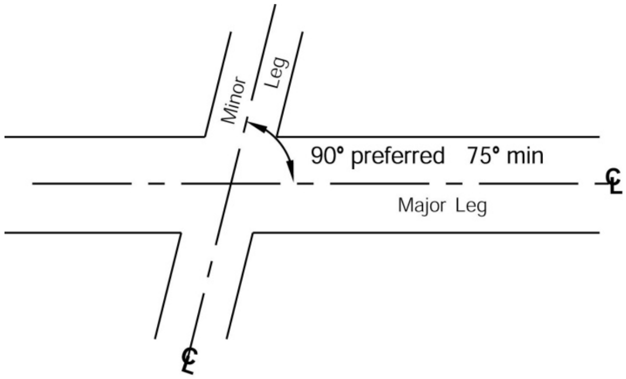
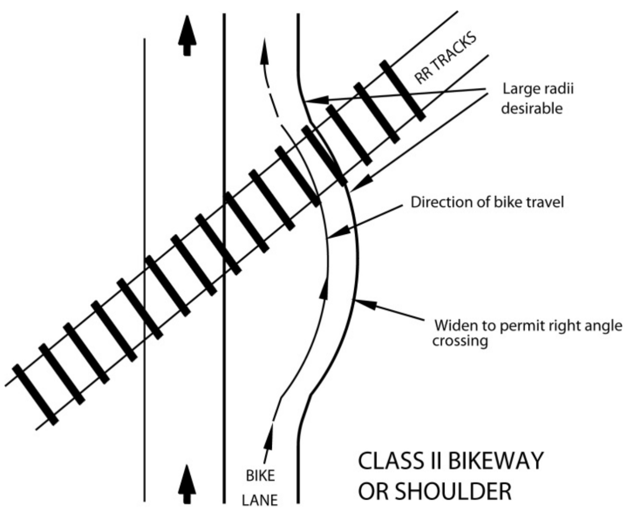


Figure 403.3.B
Class II Bikeway
Crossing Railroad



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.

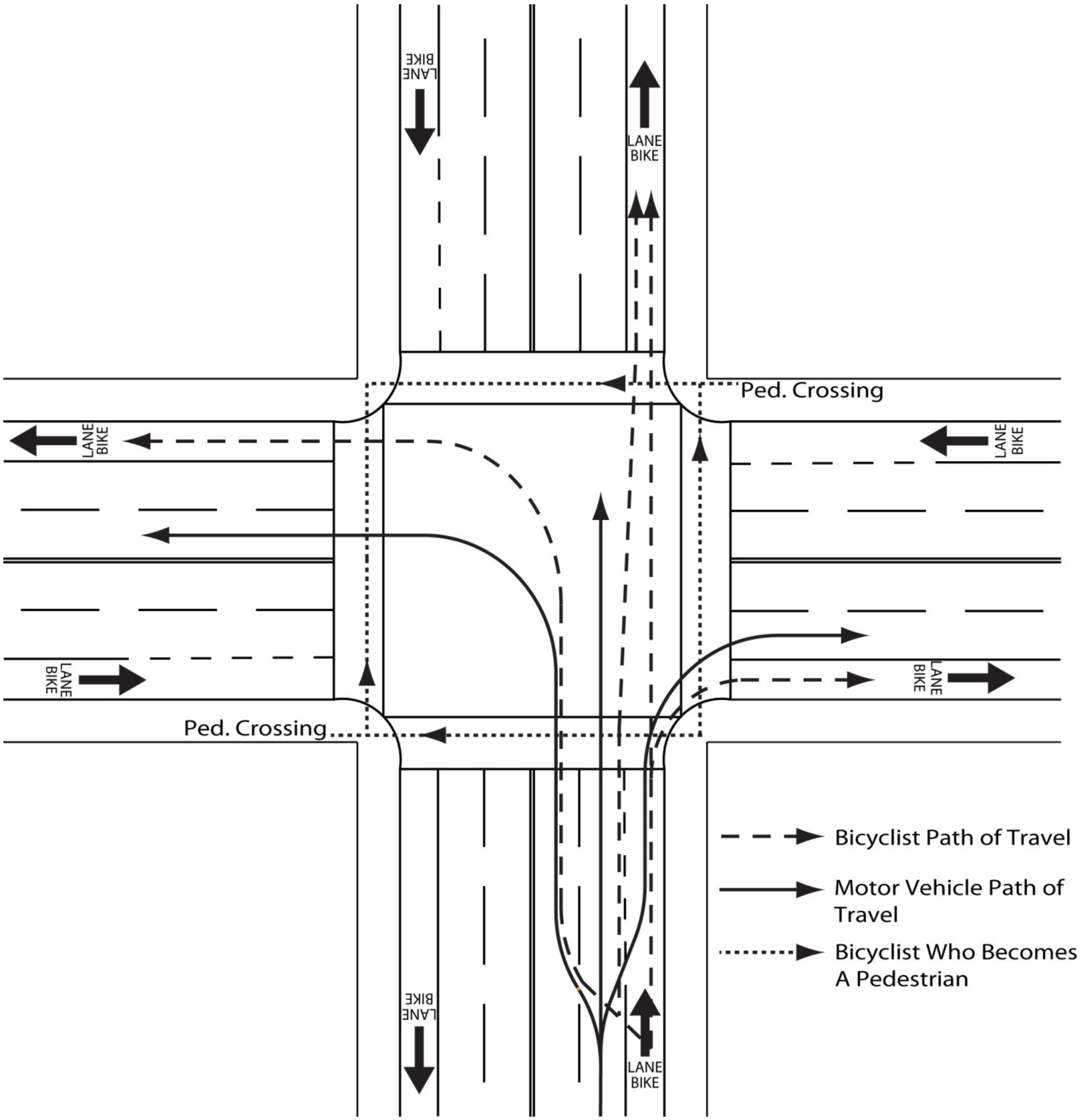
(1) *Treatment of Intersections with Right-Turn-Only Lanes.* Most motor vehicle/bicycle collisions occur at intersections. For this reason, intersection design should be accomplished in a manner that will minimize confusion by motorists and bicyclists, eliminate ambiguity and induce all road users to 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.

    <u>Optional right-turn lanes should not be used in combination with right-turn-only lanes on roads where bicycle travel is permitted.</u> 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

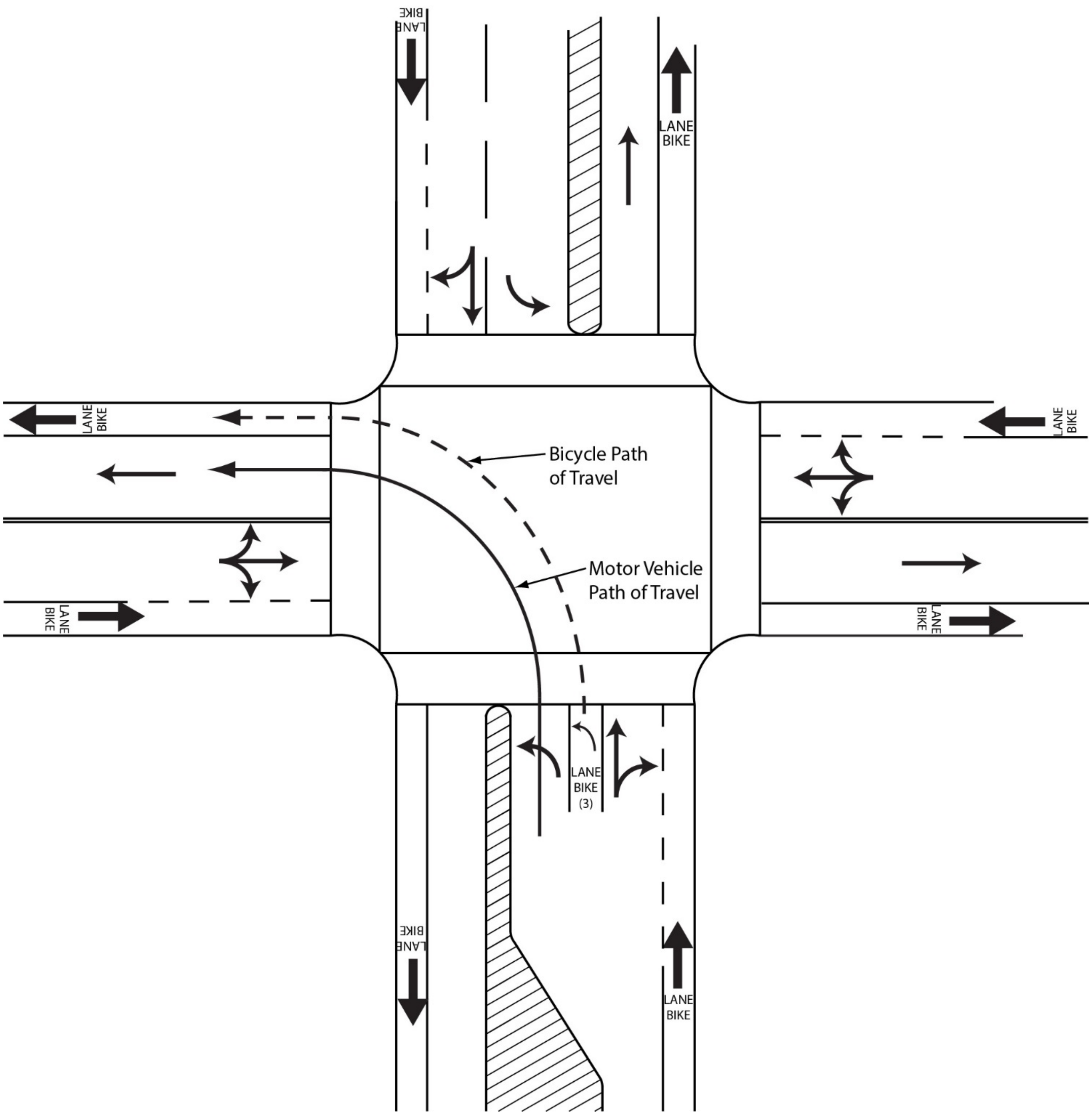


NOTE:

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, except where posted speed is greater than 40 mph, the minimum width should be 6 feet. 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 the 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 Operations and the local agency before a turning movement is not provided. The goal is to minimize possible conflicts between vehicles, bicycles, pedestrians, and other users of the roadway, while providing the minimum curb radii appropriate for the given situation.

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 Truck Managers should be consulted early in the project to ensure compliance with the design vehicle guidance contained in Topic 404. Consult local agencies to verify the location of local truck routes. 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 Route Map," indicating the National Network routes and the Terminal Access routes is posted on the Department's Office of Commercial Vehicle Operations website and is also available in printed form.

(b) STAA Design Vehicle. The STAA design vehicle is a truck tractor-semitrailer combination with a 48-foot semitrailer, a 43-foot kingpin-to-rear-axle (KPRA) distance, an 8.5-foot body and axle width, and a 23-foot truck tractor wheelbase. 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.

<u>The STAA Design Vehicle in Figures 404.5A or B should be used on the National Network, Terminal Access, California Legal, and Advisory routes.</u>

(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 KPRA for two or more axles, a maximum 38-foot KPRA 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 KPRA. 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 KPRA Advisory Routes. Advisory routes have signs posted that state the maximum KPRA length that the route can accommodate without the vehicle offtracking outside the lane. KPRA 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 Route Map indicating the California Legal routes is posted on the Department's Office of Commercial Vehicle Operations website.

(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 KPRA 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-semitrailer-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.

<u>The California Legal Design Vehicle in Figures 404.5C and D should only be used</u>

when the STAA design vehicle is not feasible and with concurrence from the District Truck Manager.

(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 Bus 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 Commercial Vehicle Operations website.

(b) 45-Foot Bus and Motorhome Design Vehicle. The 45-Foot Bus & Motorhome Design Vehicle shown in Figure 404.5F is used by Caltrans 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 indicated on the Bus 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.

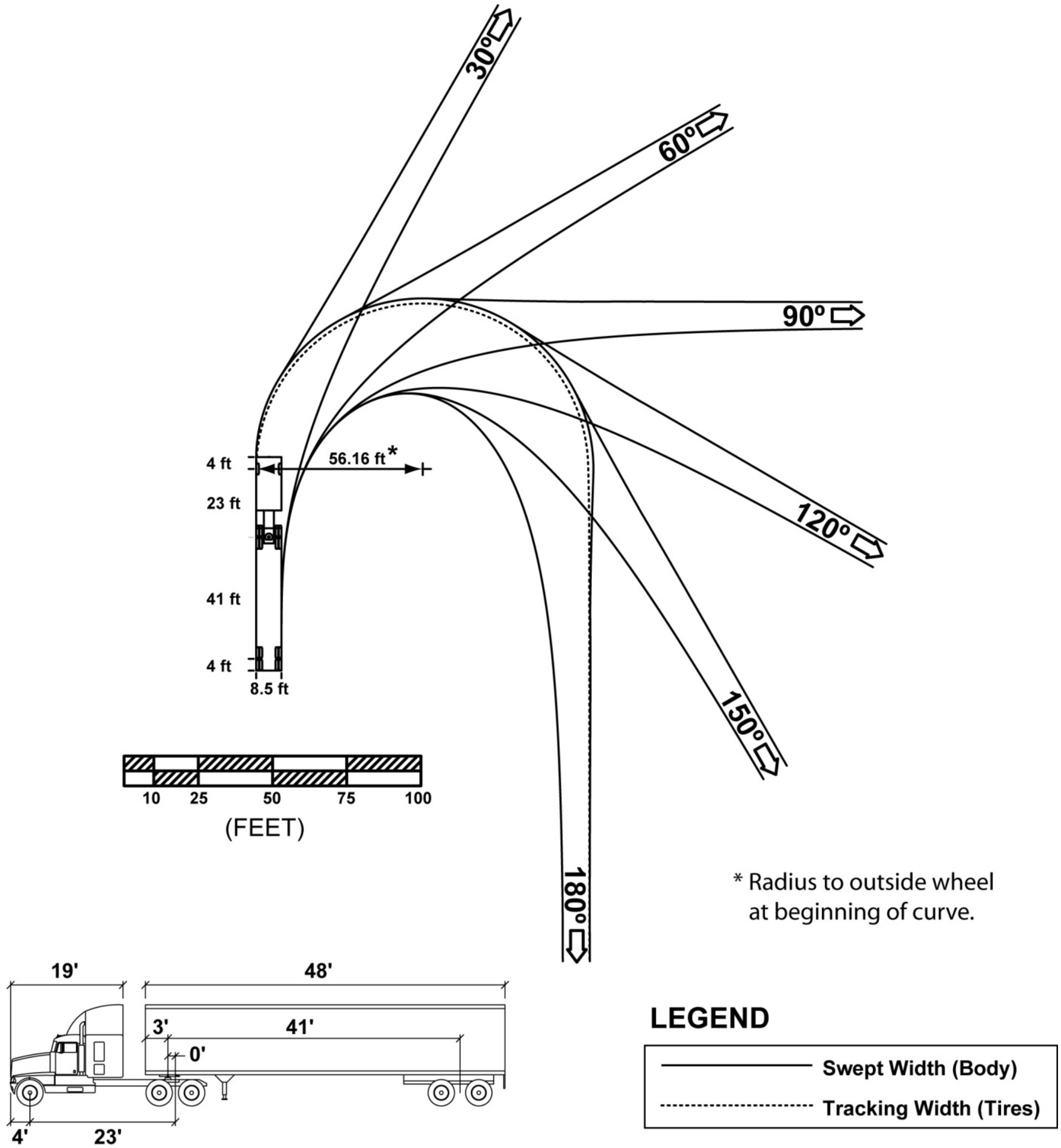
The values given in Table 405.1A provide 7-1/2 seconds for the driver on the crossroad to complete the necessary maneuver while the approaching vehicle travels at the assumed design speed of the main highway. The 7-1/2 second criterion is normally applied to all lanes of through traffic in order to cover 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 3and 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 may be excessive. High costs may be attributable to right of way acquisition, building removal, extensive excavation, or immitigable environmental impacts. In such cases a lesser value of corner sight distance, as described under the following headings, may be used.

(b) Public Road Intersections (Refer to Topic 205)--At unsignalized public road intersections (see Index 405.7) corner sight distance values given in Table 405.1A should be provided.

Figure 404.5A
STAA Design Vehicle
56-Foot Radius



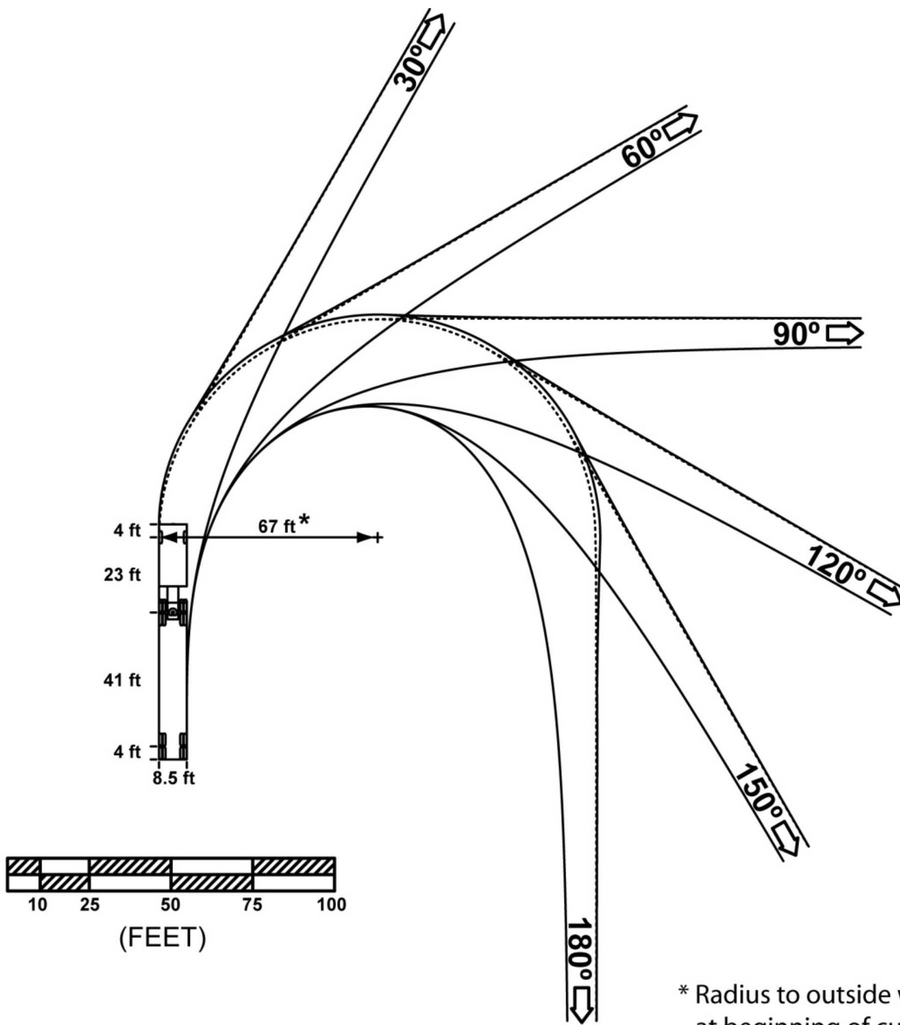
STAA - STANDARD

Tractor Width	**: 8.5'**	**Lock to Lock Time**	**: 6 seconds**
Trailer Width	**: 8.5'**	**Steering Lock Angle**	**: 26.3 degrees**
Tractor Track	**: 8.5'**	**Articulating Angle**	**: 70 degrees**
Trailer Track	**: 8.5'**		

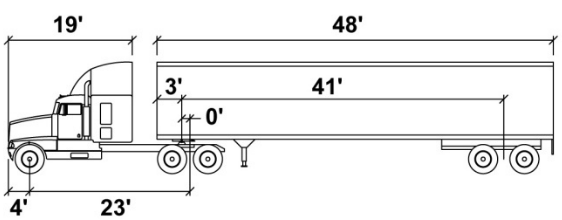
Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5B

STAA Design Vehicle
67-Foot Radius



* Radius to outside wheel at beginning of curve.



LEGEND

Swept Width (Body)
Tracking Width (Tires)

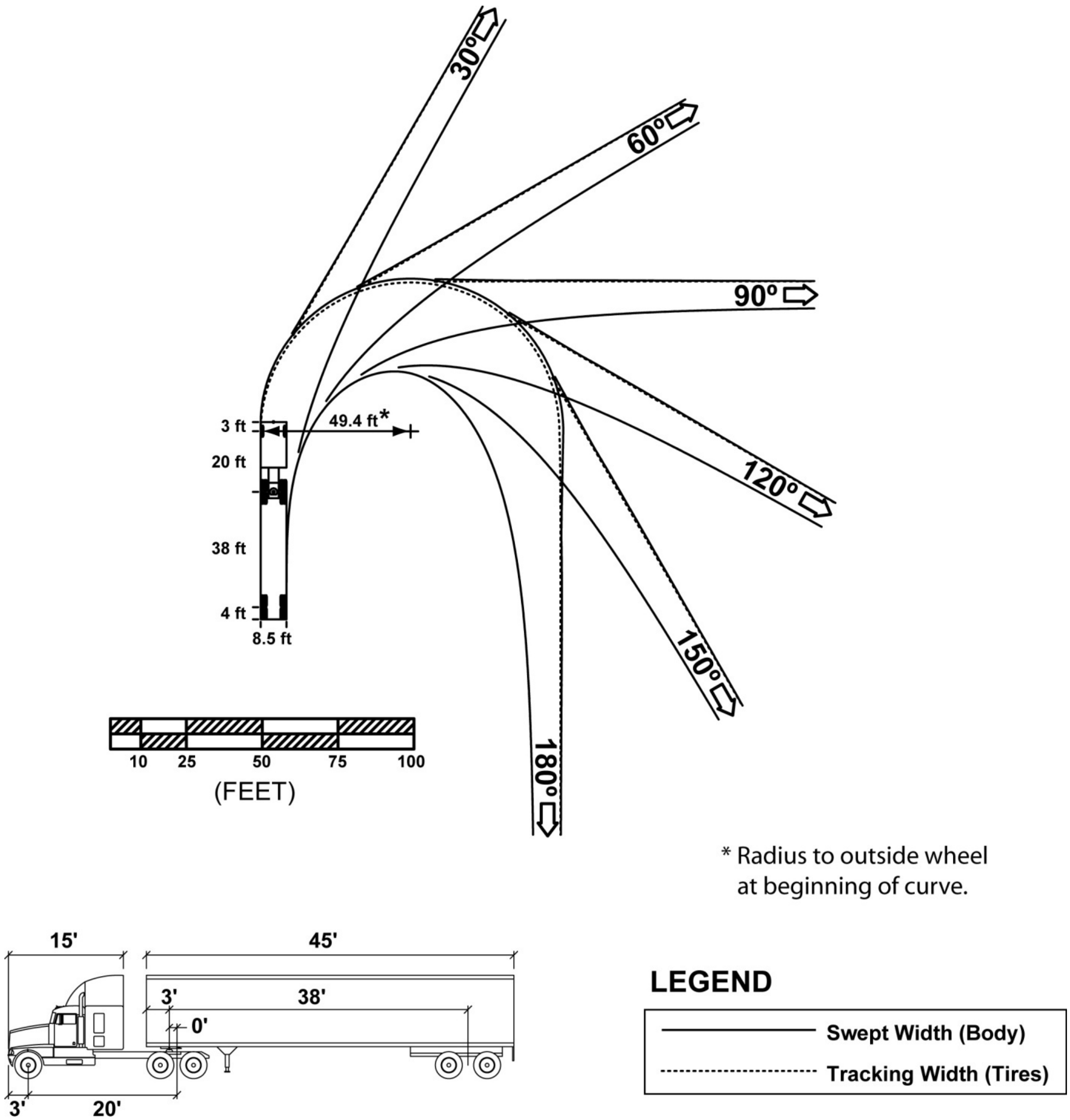
STAA - STANDARD

Tractor Width	: 8.5'	Lock to Lock Time	: 6 seconds
Trailer Width	: 8.5'	Steering Lock Angle	: 26.3 degrees
Tractor Track	: 8.5'	Articulating Angle	: 70 degrees
Trailer Track	: 8.5'		

Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5C

California Legal Design Vehicle
50-Foot Radius



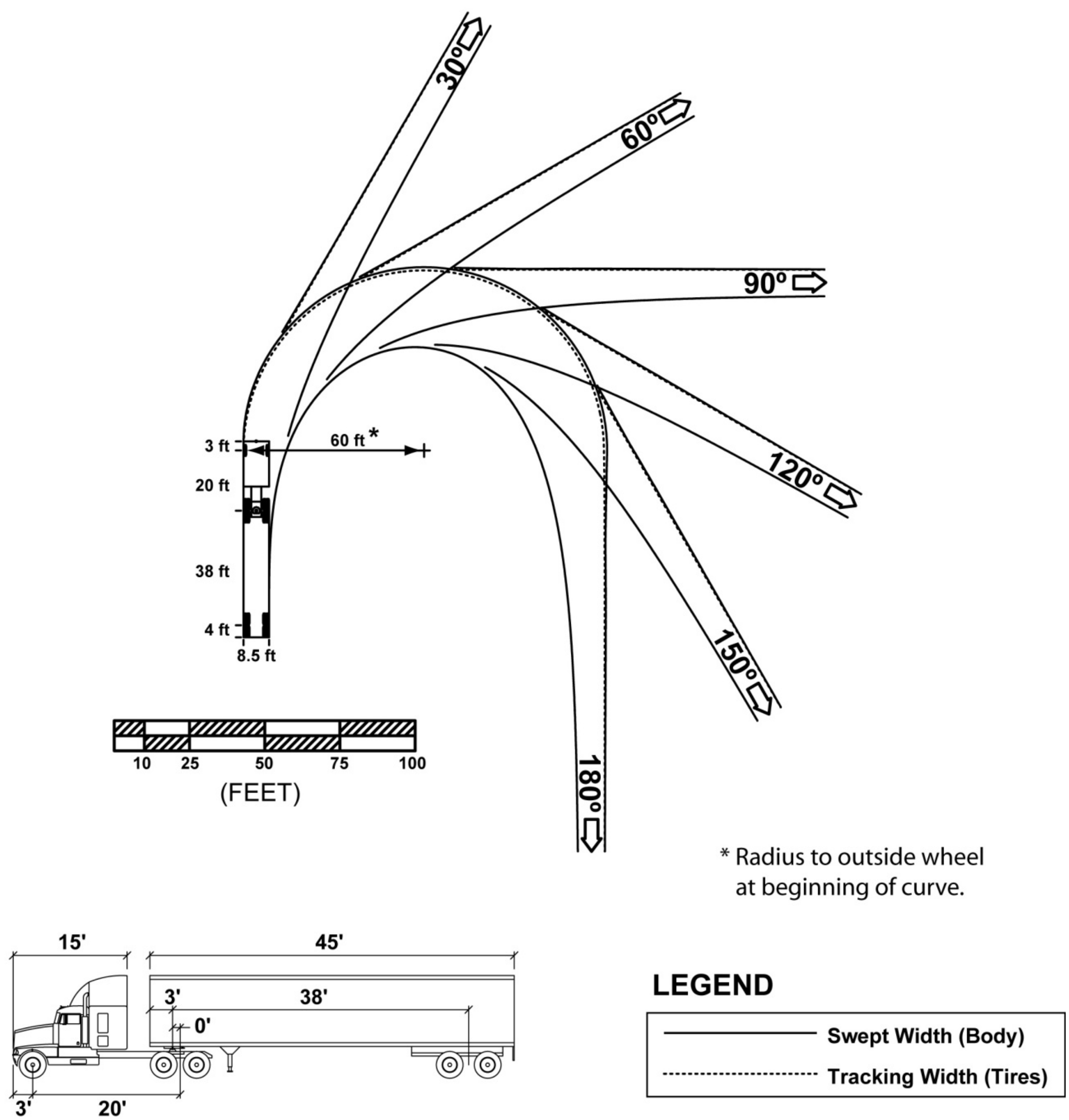
CA LEGAL - 65 FT

Tractor Width	: 8.5'	Lock to Lock Time	: 6 seconds
Trailer Width	: 8.5'	Steering Lock Angle	: 26.3 degrees
Tractor Track	: 8.5'	Articulating Angle	: 70 degrees
Trailer Track	: 8.5'		

Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5D

California Legal Design Vehicle
60-Foot Radius



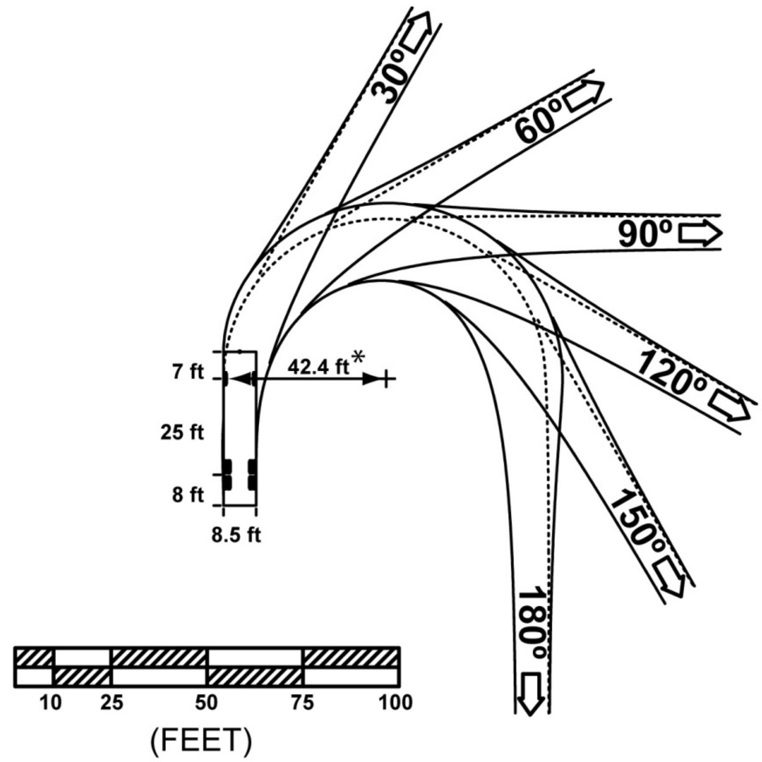
CA LEGAL - 65 FT

Tractor Width	**: 8.5'**	**Lock to Lock Time**	**: 6 seconds**
Trailer Width	**: 8.5'**	**Steering Lock Angle**	**: 26.3 degrees**
Tractor Track	**: 8.5'**	**Articulating Angle**	**: 70 degrees**
Trailer Track	**: 8.5'**		

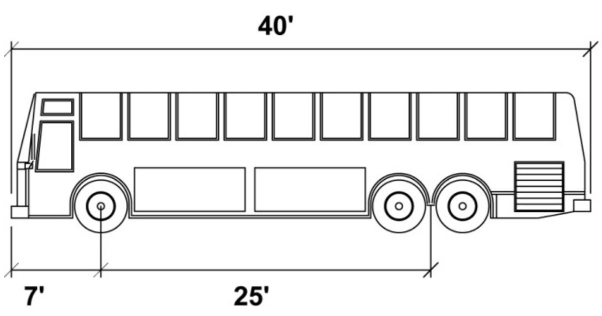
Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5E

40-Foot Bus Design Vehicle



* Radius to outside wheel
at beginning of curve.



LEGEND

————————	**Swept Width (Body)**
------------------------	**Tracking Width (Tires)**

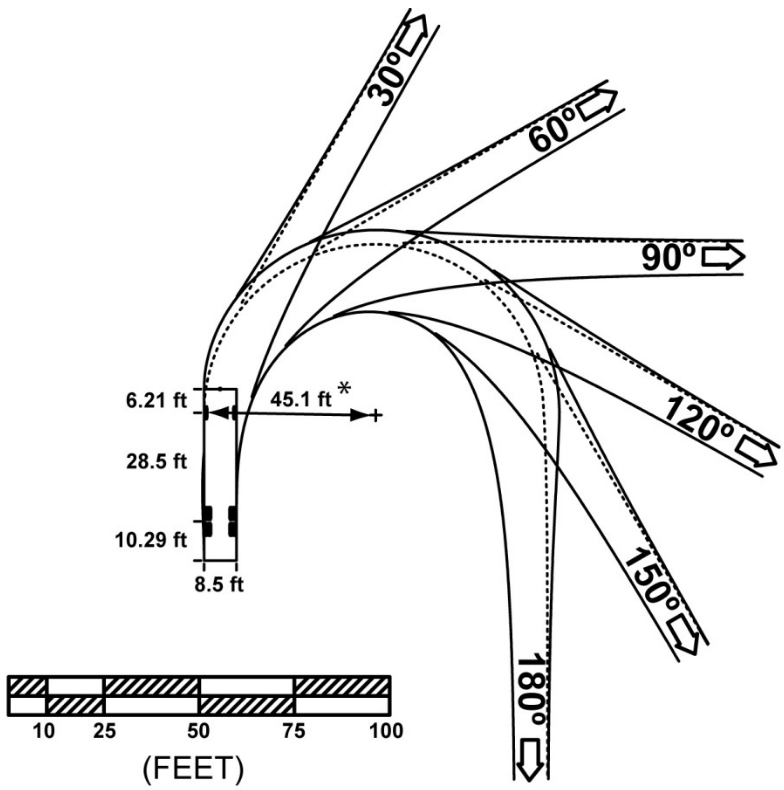
40' BUS

Width : 8.5'
Track : 8.5'
Lock to Lock Time : 6 seconds
Steering Lock Angle: 41.0 degrees

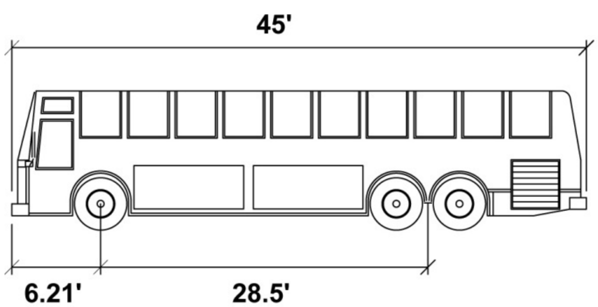
Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5F

45-Foot Bus & Motorhome Design Vehicle



* Radius to outside wheel at beginning of curve.



LEGEND

————————	Swept Width (Body)
- - - - - - - - - -	Tracking Width (Tires)

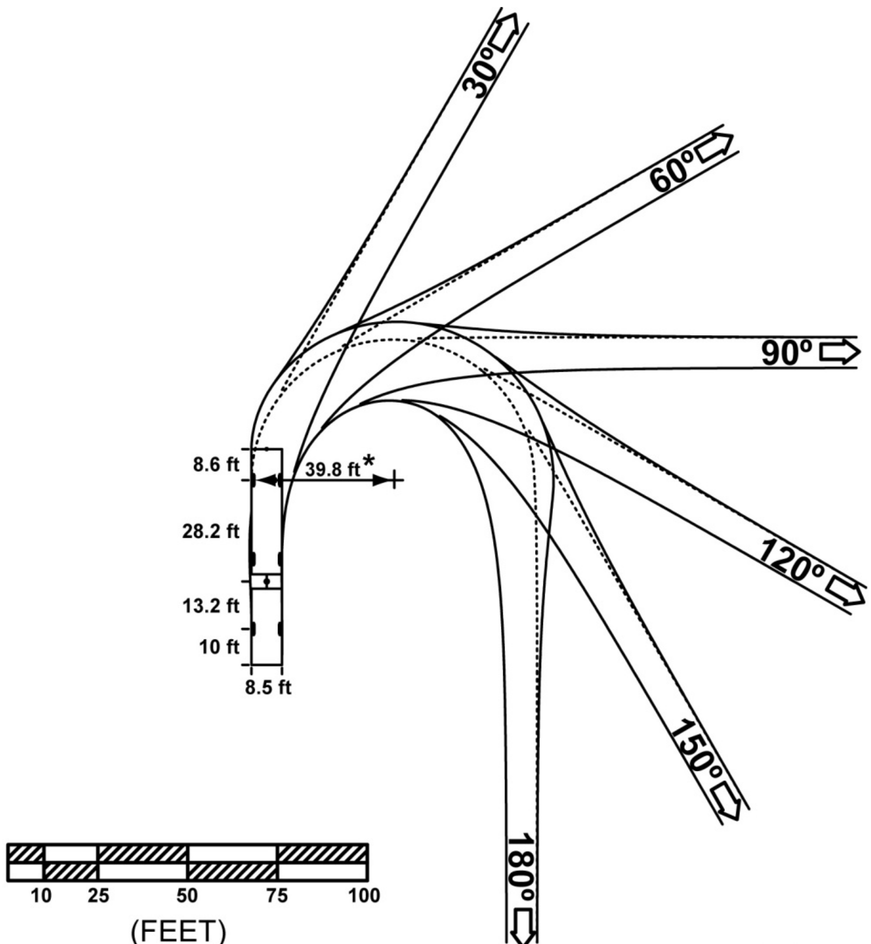
45' BUS

Width	**: 8.5'**
Track	**: 8.5'**
Lock to Lock Time	**: 6 seconds**
Steering Lock Angle	**: 44.3 degrees**

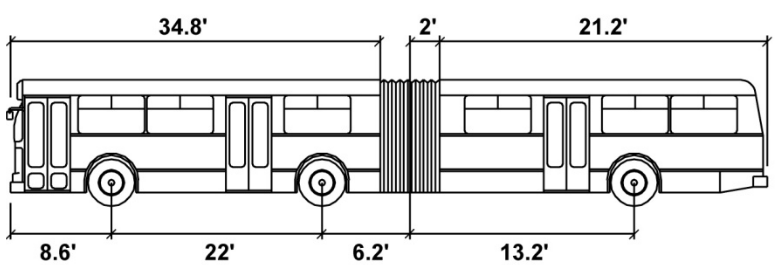
Note: For definitions, see Indexes 404.1, and 404.5.

Figure 404.5G

60-Foot Articulated Bus Design Vehicle



* Radius to outside wheel at beginning of curve.



LEGEND

————	Swept Width (Body)
- - - - - -	Tracking Width (Tires)

ARTICULATED BUS

Width	: 8.5'
Track	: 8.5'
Lock to Lock Time	: 6 seconds
Steering Lock Angle	: 38.3 degrees
Articulating Angle	: 50.0 degrees

Note: For definitions, see Indexes 404.1 and 404.5.

At signalized intersections the values for corner sight distances given in Table 405.1A should also be applied whenever possible. Even though traffic flows are designed to move at separate times, unanticipated conflicts can occur due to violation of signal, right turns on red, malfunction of the signal, or use of flashing red/yellow mode.

Table 405.1A
Corner Sight Distance
(7-1/2 Second Criteria)

Design Speed (mph)	Corner Sight Distance (ft)
25	275
30	330
35	385
40	440
45	495
50	550
55	605
60	660
65	715
70	770

Where restrictive conditions exist, similar to those listed in Index 405.1(2)(a), the minimum value for corner sight distance at both signalized and unsignalized intersections shall be equal to the stopping sight distance as given in Table 201.1, measured as previously described.

(c) Private Road Intersections (Refer to Index 205.2) and Rural Driveways (Refer to Index 205.4)--**The minimum corner sight distance shall be equal to the stopping sight distance as given in Table 201.1, measured as previously described.**

(d) Urban Driveways (Refer to Index 205.3)--Corner sight distance requirements as described above are not applied to urban driveways.

(3) *Decision Sight Distance.* At intersections where the State route turns or crosses another State route, the decision sight distance values given in Table 201.7 should be used. In computing and measuring decision sight distance, the 3.5-foot eye height and the 0.5-foot object height should be used, the object being located on the side of the intersection nearest the approaching driver.

The application of the various sight distance requirements for the different types of intersections is summarized in Table 405.1B.

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).

(4) *Acceleration Lanes for Turning Moves onto State Highways.* At rural intersections, with "STOP" control on the local cross road, acceleration lanes for left and right turns onto the State facility should be considered. At a minimum, the following features should be evaluated for both the major highway and the cross road:

- divided versus undivided
- number of lanes

- design speed
- gradient
- lane, shoulder and median width
- traffic volume and composition of highway users, including trucks and transit vehicles
- turning volumes
- horizontal curve radii
- sight distance
- proximity of adjacent intersections
- types of adjacent intersections

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

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.

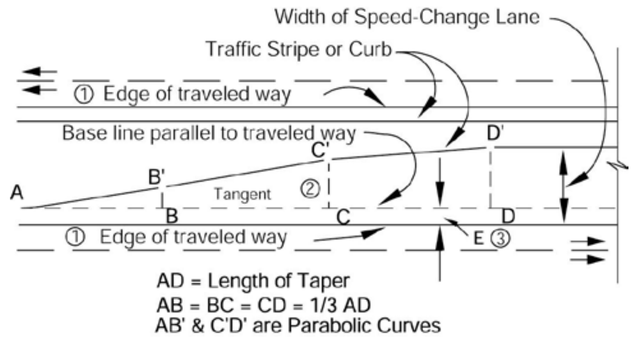
(d) Deceleration Lane Length -- Design speed of the roadway approaching the intersection should be the basis for determining deceleration lane length. It is desirable that deceleration take place entirely off the through traffic lanes. Deceleration lane lengths are given in Table 405.2B; the bay taper 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 inter-sections, 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.

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.

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	40.0	2.50	2.75	3.00	B'
	30	45.0	60.0	5.00	5.50	6.00	
C'	40	60.0	80.0	7.50	8.25	9.00	C'
	45	67.5	90.0	8.59	9.45	10.31	
	50	75.0	100.0	9.38	10.31	11.25	
	55	82.5	110.0	9.84	10.83	11.81	
	60	90.0	120.0	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

(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.

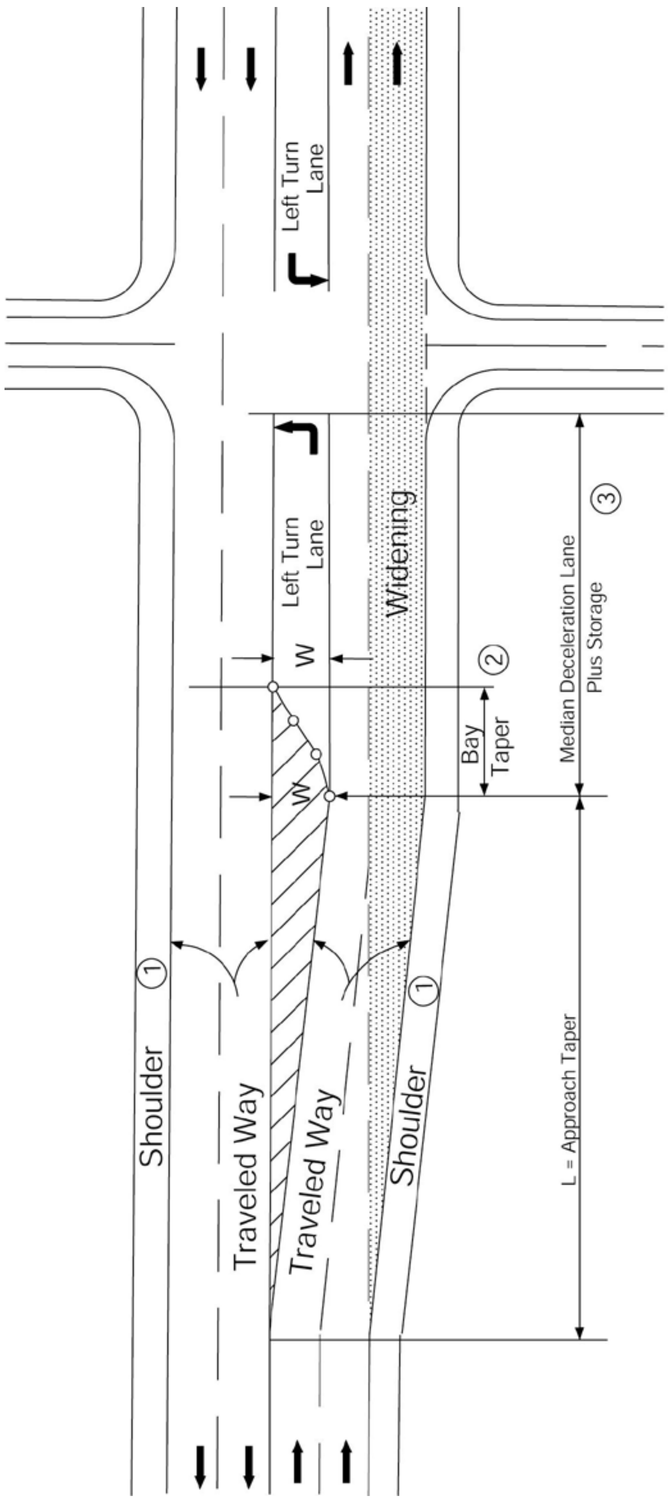
(2) *Design Elements.*

(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.

  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.

Figure 405.2A
Standard Left-turn Channelization



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

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

NOTES:

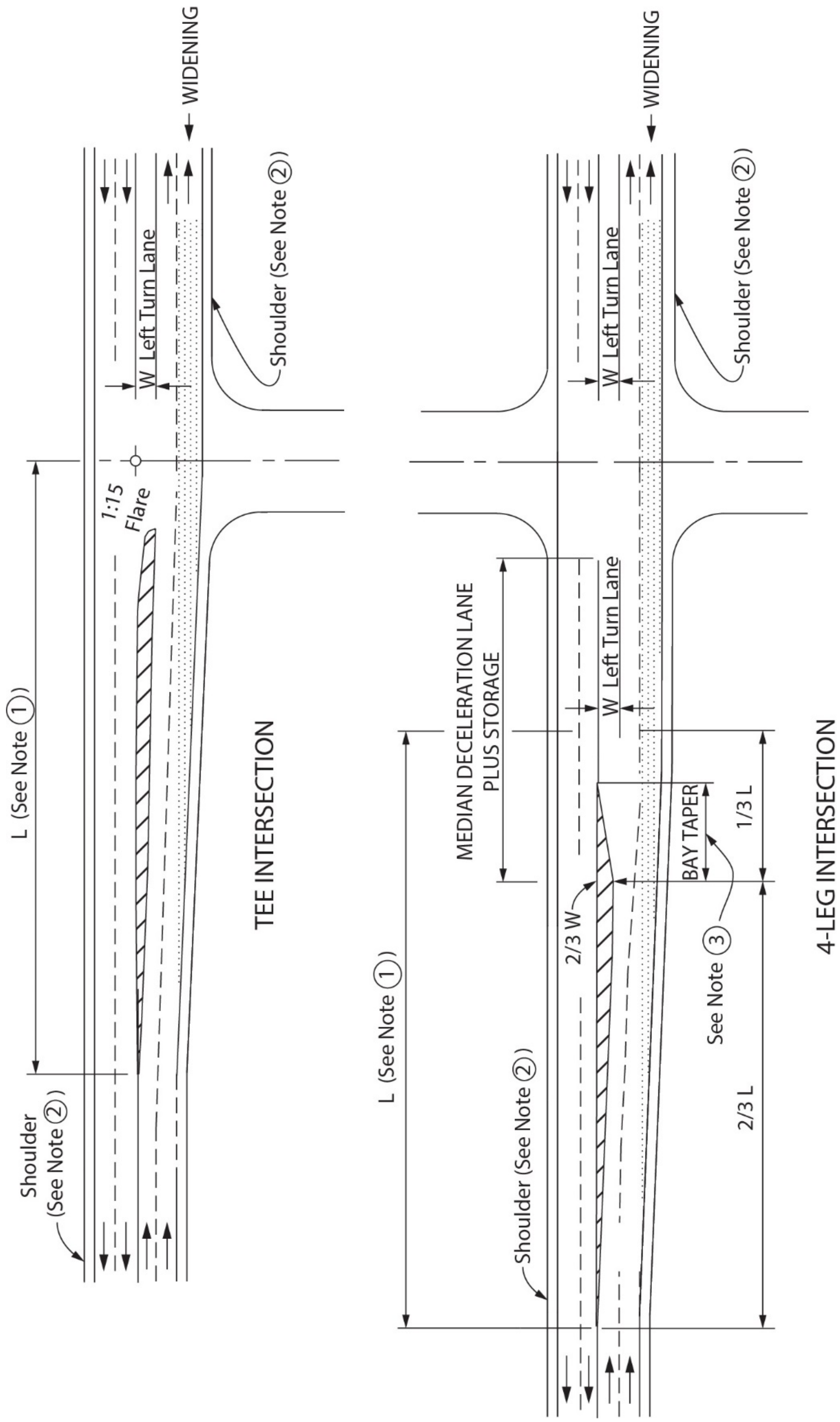
(1) Where width is restricted, shoulder width may be reduced and parking restricted with an approved design exception pursuant to Index 82.2.
For bicycle use, a minimum 4-foot shoulder is required (5-foot if gutter is present).

(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)

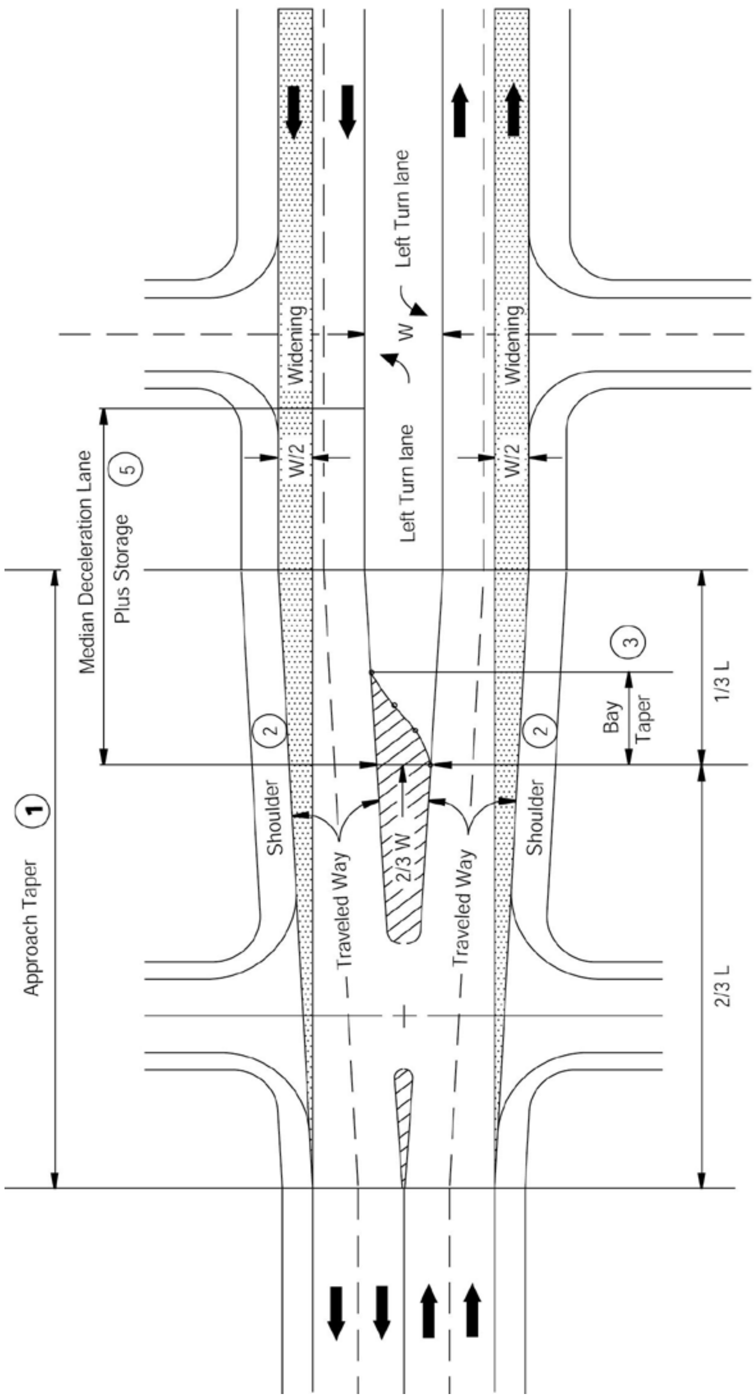
EQUATION

$$L = \begin{cases} WV, \text{ for } V \geq 45\text{mph} \\ \text{Or } WV^2/60, \text{ for } V < 45\text{mph} \end{cases}$$

Where:

L = Length of Transition - feet
W = Width of Median Lane - feet
V = Design Speed - mph

Figure 405.2C
Minimum Median Left-turn Channelization
(Widening on Both Sides in Urban Areas with Short Blocks)



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 feet shoulder is required (5 feet if gutter is present).
3. Bay taper length = 60 feet to 120 feet. (See Table 405.2A)
4. Assumes equal widening each side. Where widening is unequal, use a fraction that is proportional to widening on each side.
5. For deceleration lane length see Table 405.2B.

EQUATION: 4

L = Use (1/2)WV, for $V \geq 45$ mph
Or $WV^2/120$, for $V < 45$ mph

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

(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.

<u>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.</u> 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

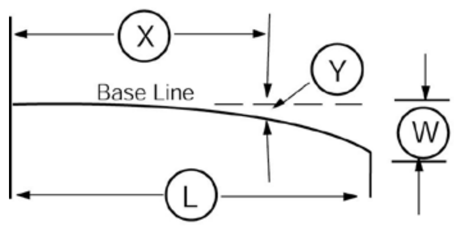
(1) *General.* Median openings, sometimes called crossovers, provide for crossings of the median at designated locations. Except for emergency passageways in a median barrier, median openings are not allowed on urban freeways.

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.

(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

Table 405.4

Parabolic Curb Flares Commonly Used



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

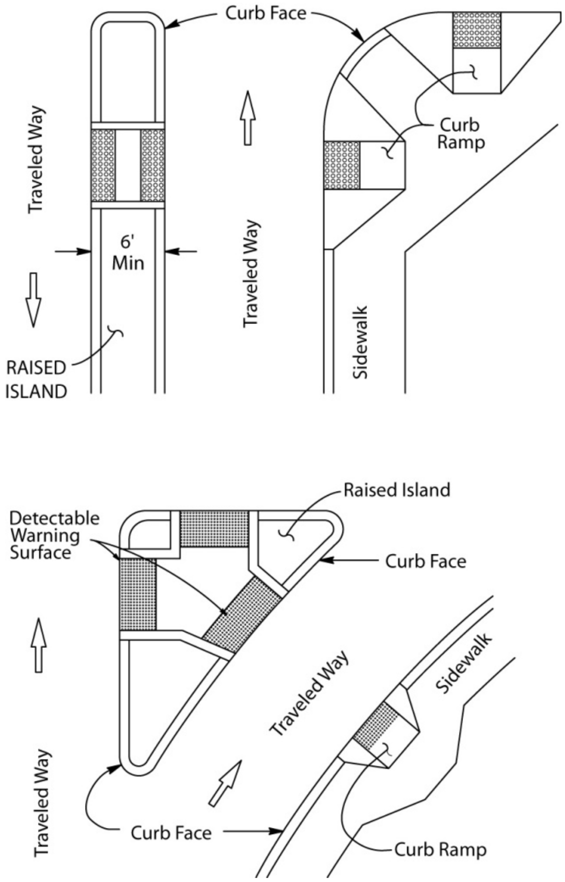
L = Length of flare in feet
W = Maximum offset in feet
X = Distance along base line in feet
Y = Offset from base line in feet

W is shown in table thus

OFFSET IN FEET FOR GIVEN "X" DISTANCE																
Distance (X) / (L) Length of Flare	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



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.

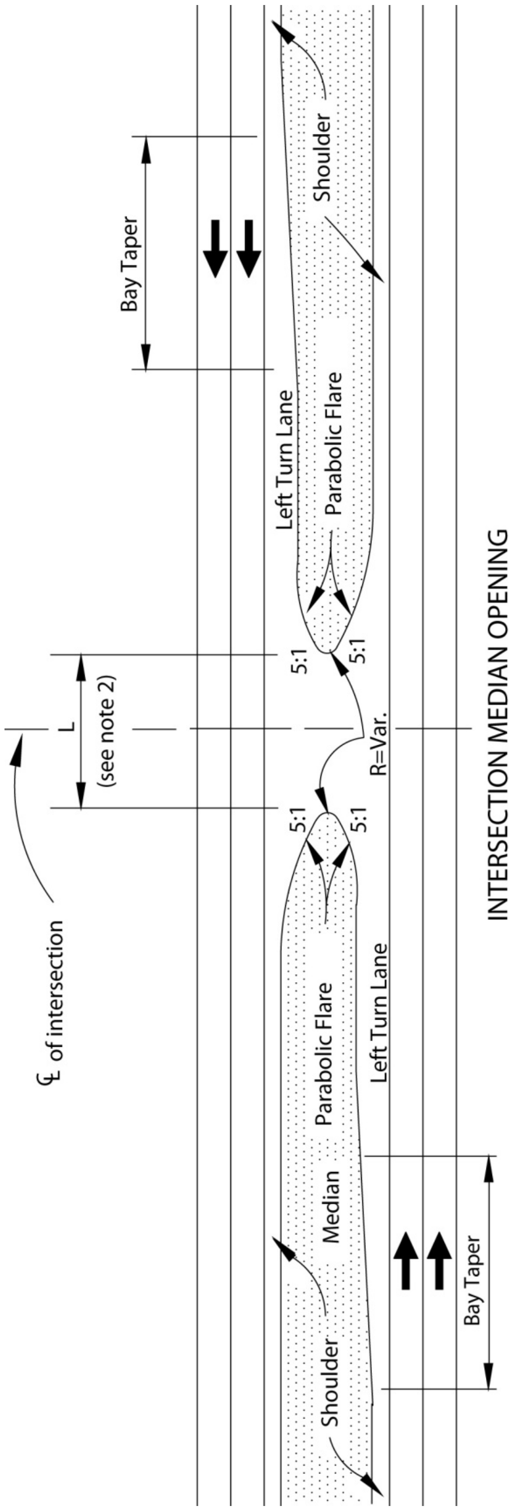
405.6 Access Control

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. 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.

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 Pedeistrain and bicycle features are not shown on figure.

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

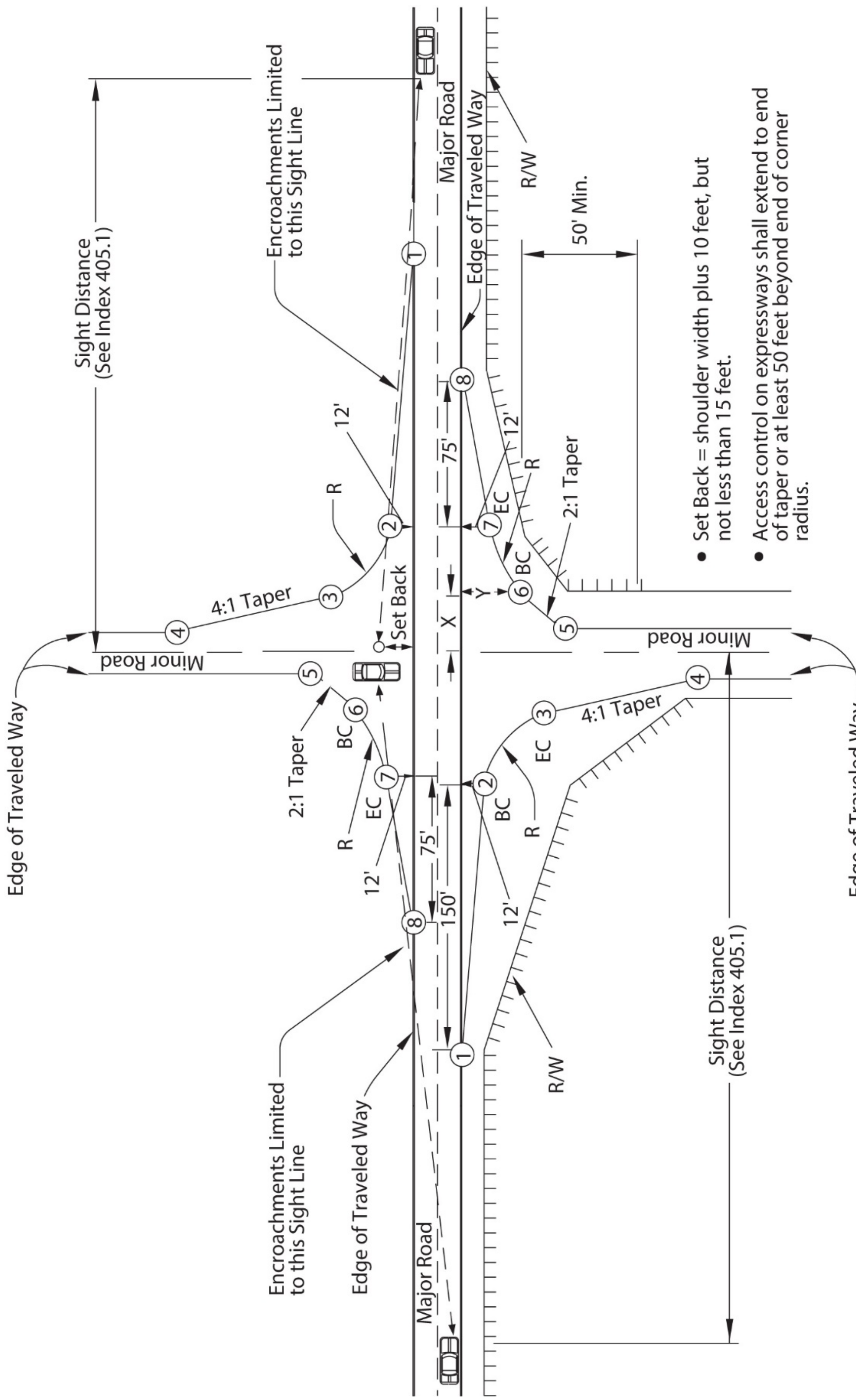
Two-lane State highways may be widened at intersections to 4-lanes whenever signals are installed. Sometimes it may be necessary to widen the intersecting road. The minimum design is shown in Figure 405.9. More elaborate treatment may be warranted by the volume and pattern of traffic movements. Unusual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

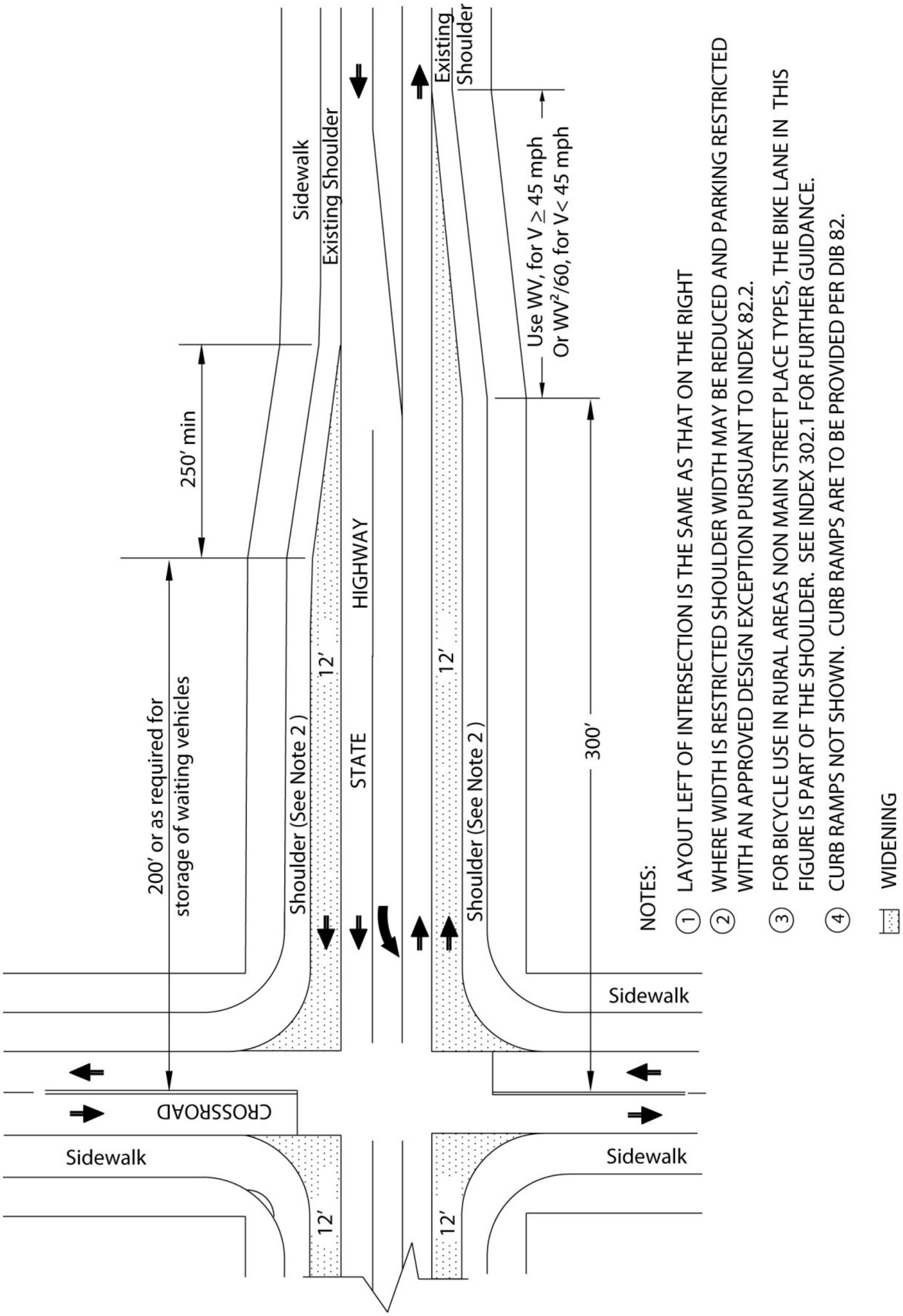
Figure 405.7
Public Road Intersections



X - Distance measured from centerline of minor road along major road - feet.
Y - Offset distance measured from edge of traveled way of major road to any given point - feet.

Radius of Curve	Design Vehicle	Pt 1 X	Pt 1 Y	Pt 2 X	Pt 2 Y	Pt 3 X	Pt 3 Y	Pt 4 X	Pt 4 Y	Pt 5 X	Pt 5 Y	Pt 6 X	Pt 6 Y	Pt 7 X	Pt 7 Y	Pt 8 X	Pt 8 Y
30'	Bus	204.20	0.0	54.20	12.0	27.49	34.63	12.0	96.58	12.0	40.66	18.23	28.21	40.32	12.0	115.32	0.0
40'	California	215.08	0.0	65.08	12.0	29.46	42.17	12.0	112.03	12.0	53.35	21.87	33.61	51.33	12.0	126.33	0.0
50'	STAA	226.09	0.0	76.09	12.0	31.57	49.71	12.0	127.98	12.0	75.63	30.31	39.01	67.13	12.0	142.13	0.0

Figure 405.9
Widening of Two-lane Roads at Signalized Intersections



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.

First consider the design of a single lane roundabout per the design period guidance in Index 103.2. If a second lane is not needed until 10 or more years, it may be better to phase the improvements. Construct the first phase of the roundabout so at the 20-year design period, an additional lane can be easily added. In order to comply with the 10-year design period guidance provided in Index 103.2, the initial project must provide the right of way needed for utility relocations, a shared-use path designed for a Class I Bikeway, and all other features other than pavement, lighting, and striping in their ultimate locations.

In some locations, it may not be practical to build a single lane roundabout that will operate for 10 years. Geometric constraints and other conflicts may preclude widening to the ultimate configuration. In such cases, other intersection configurations or control strategies addressed in Index 401.5 may need to be considered.

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.

<u>The following entry speeds should not be exceeded</u>:

- <u>Single lane roundabouts, 25 mph.</u>
- <u>Multilane roundabouts, 30 mph.</u>

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 detectable warning surface (truncated domes) differentiates a pedestrian curb ramp from a bicycle ramp.
- Detectable warning surface is 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.

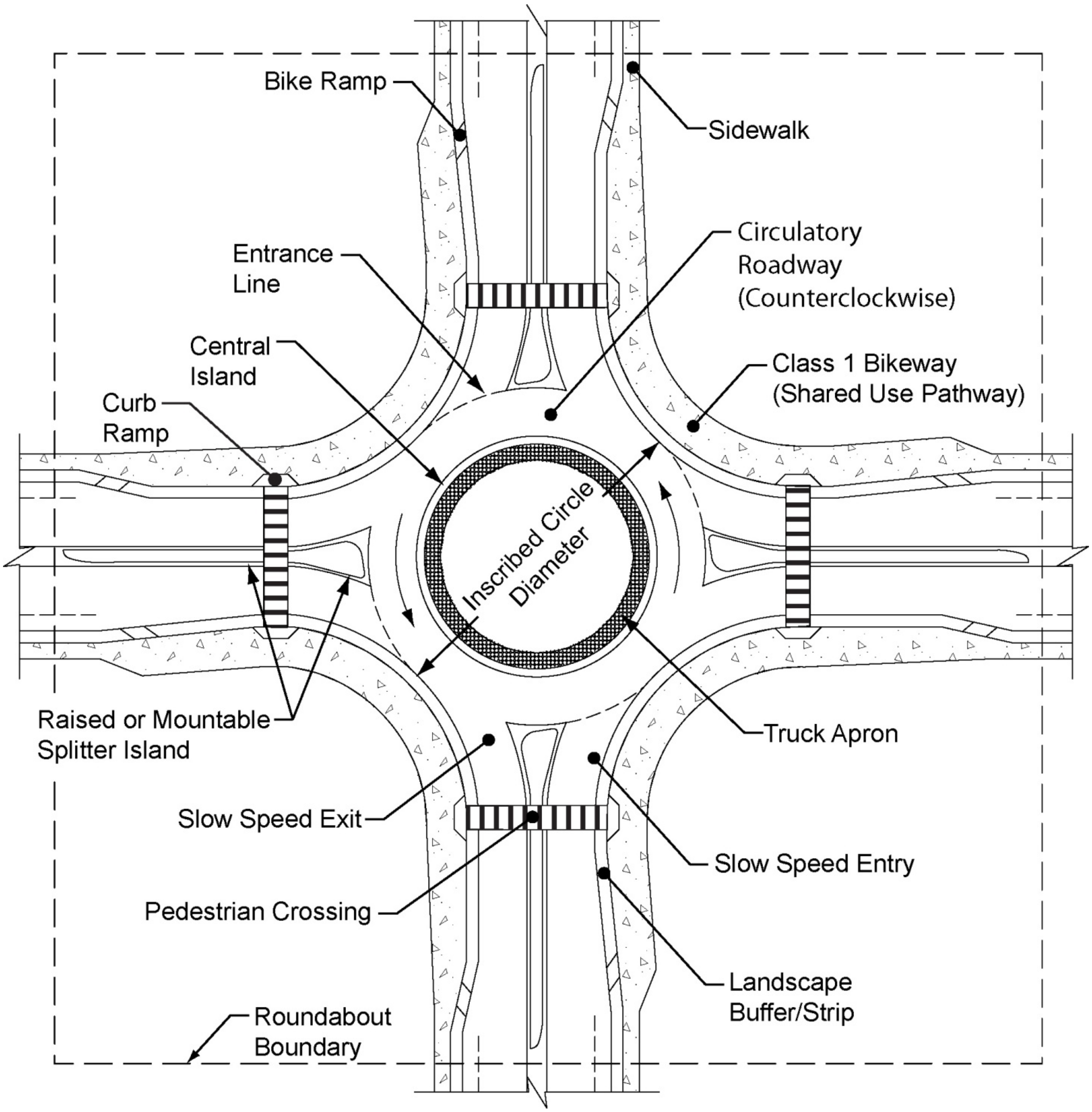
(b) Bicyclist Use of the Circular Roadway. Single lane roundabouts do not require bicyclists to change lanes in the circular roadway to select the appropriate lane for their direction of travel, so they tend to be comfortable for bicyclists to use. Even two-lane roundabouts, which may have straighter 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.

Figure 405.10
Roundabout Geometric Elements



NOTE:

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

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 thetransit 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.

(19) *Maintenance.*

In climate regions where snowfall occurs and the use of snow removal equipment is necessary, consider tapering the approach ends of curbs. Contact the District Maintenance Engineer and appropriate Regional Manager for maintenance strategies and practices including seasonal operations, maintenance resources, and specialized equipment. Special equipment or procedures may be needed. Maintenance responsibilities may also include multiple state, county, and city agencies where coordination of maintenance efforts and funding is needed.

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

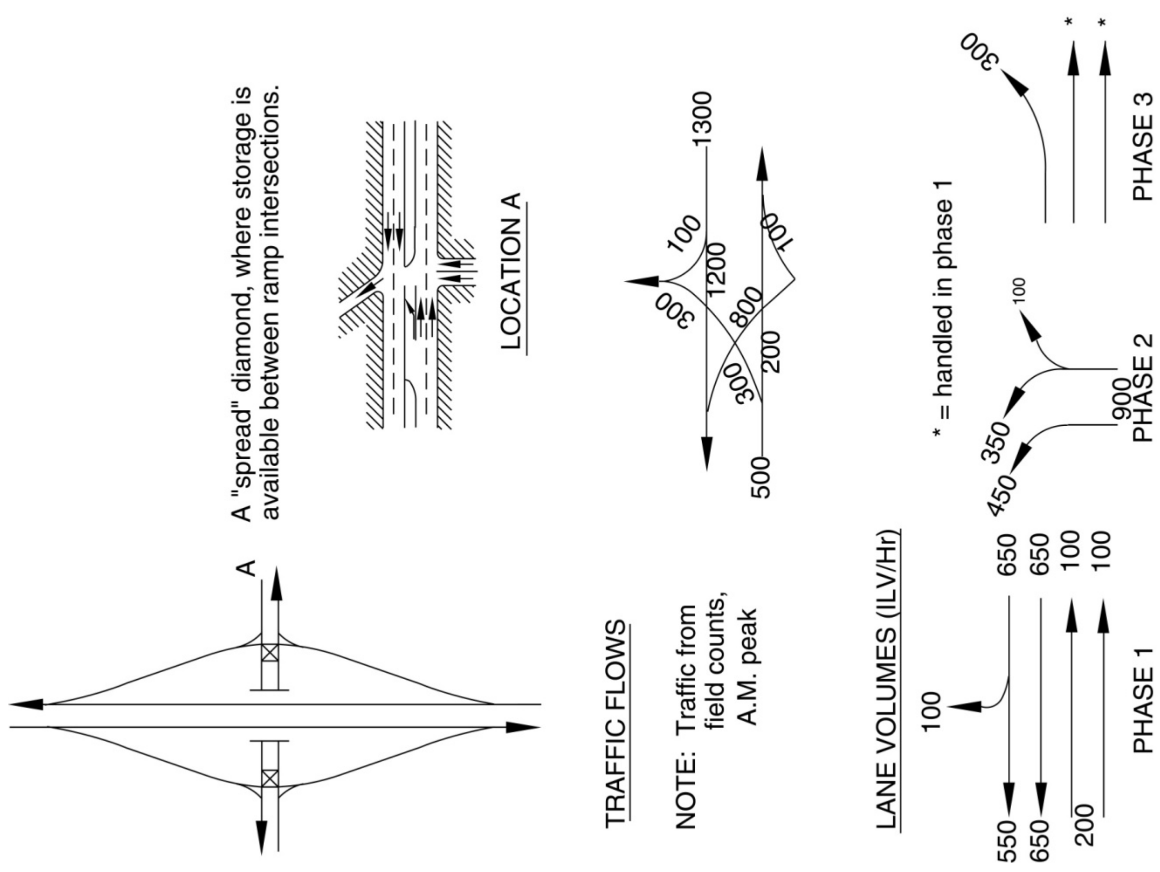
Vehicle Traffic Flow Conditions at Intersections at Various Levels of Operation

ILV/hr	Description
< 1200:	Stable flow with slight, but acceptable delay. Occasional signal loading may develop. Free midblock operations.
1200-1500:	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.
1500 (Capacity):	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.

Figure 406A
Spread Diamond



Identify and Sum-up Critical Lane Volumes

PHASE 1: 650 ~~100~~ → 650 ILV/Hr.

PHASE 2: 450 → 450 ILV/Hr.

PHASE 3: ~~*~~ 300 → 300 ILV/Hr.

Total - - - - - 1400 ILV/Hr.

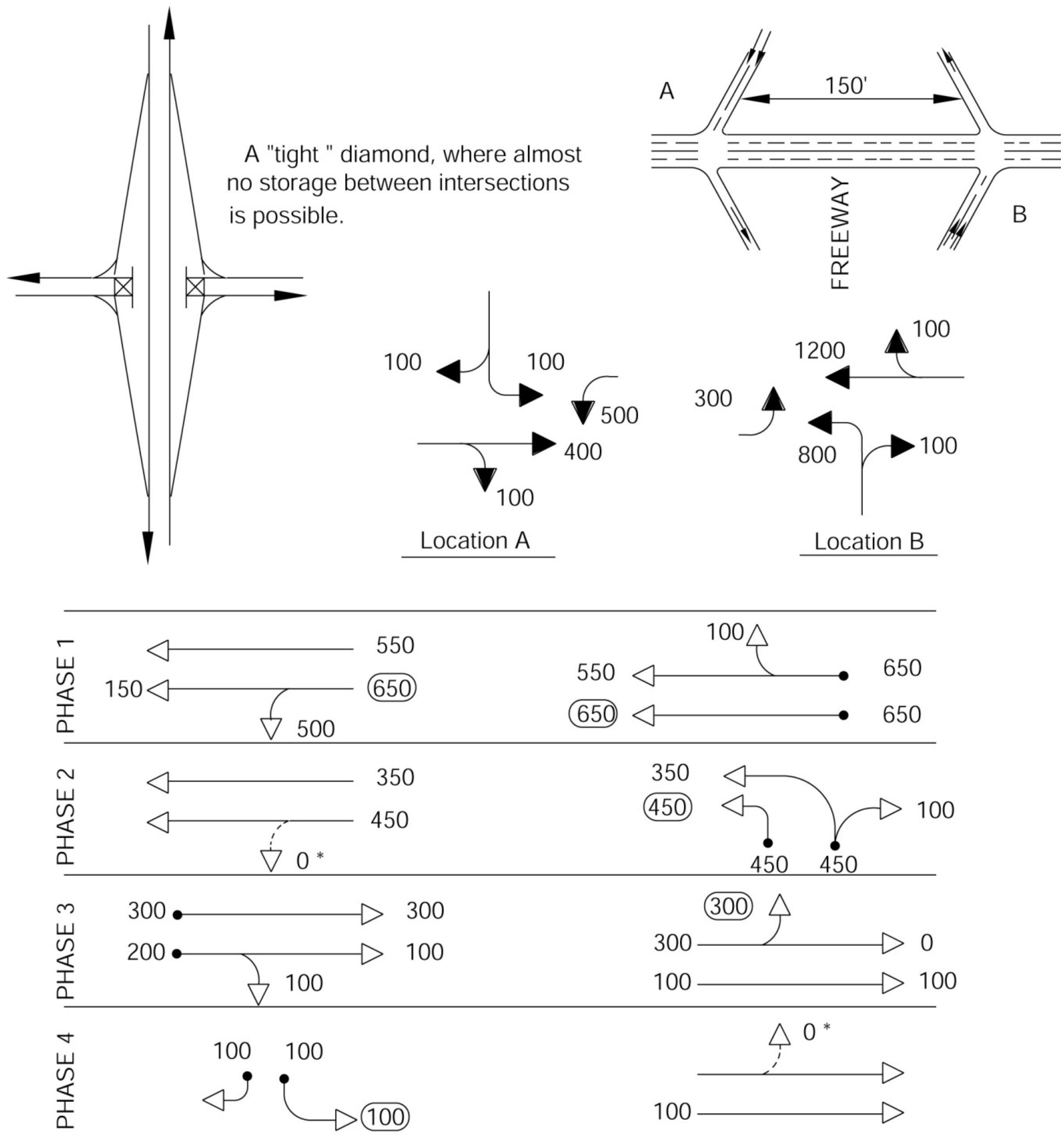
Evaluate Operating Level: (1400 ILV/Hr.)

1200 < 1400 < 1500

The total volume of traffic which shares the intersection does not exceed capacity (1500), but is greater than 1200 ILV/Hr. threshold. This suggests that congestion would be present and the intersection would be approaching capacity.

ILV = Intersecting Lane Vehicles.

Figure 406B
Tight Diamond

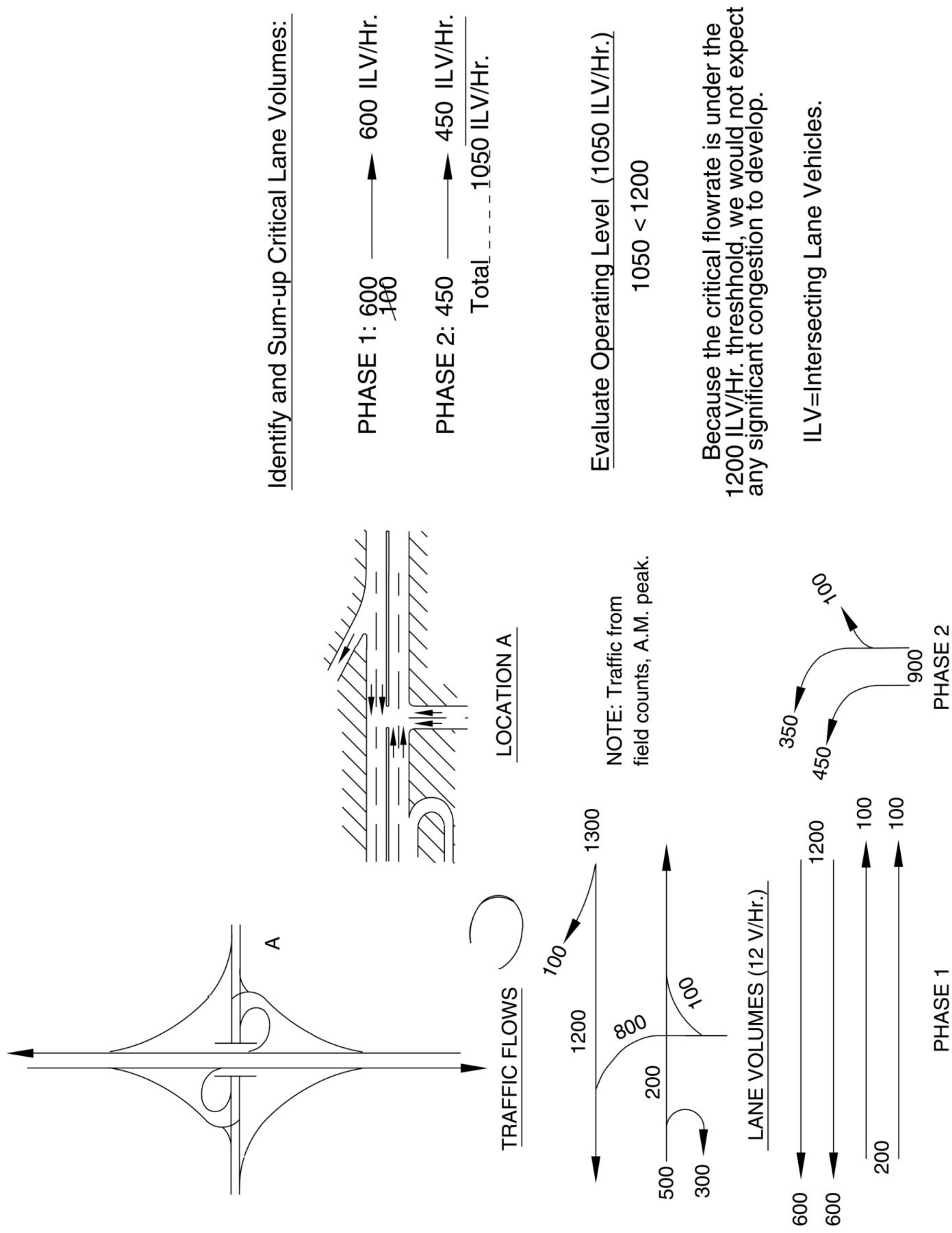


*NOTE: When no storage at all is permitted, left-turn movement is cleared during this phase.

Critical Lane Volumes:

	650
	450
	300
ILV=Intersecting Lane Vehicles.	100
	1500 ILV/Hr.

Figure 406C
Two-quadrant Cloverleaf



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 us 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.

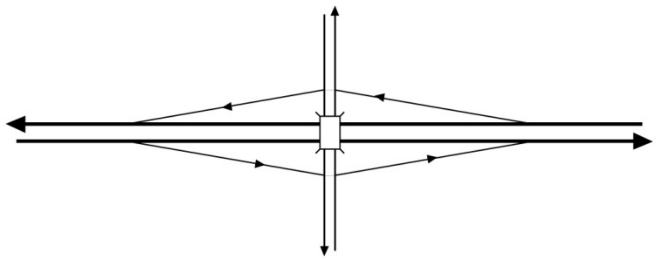
Class II bikeways designed through interchanges should be accomplished considering the mobility of bicyclists and should be designed in a manner that will minimize confusion by motorists and bicyclists. Designs which allow high speed merges at on- and off-ramps to local streets and conventional highways 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.

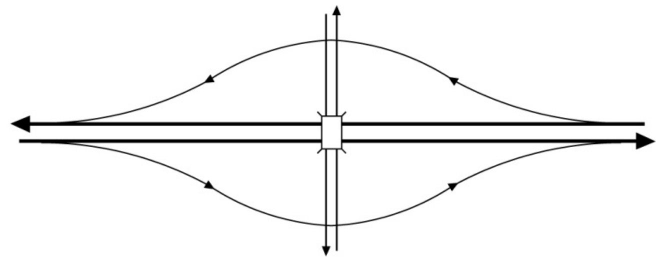
    The compact diamond (Type L-1) is most adaptable where the freeway is depressed or

Figure 502.2

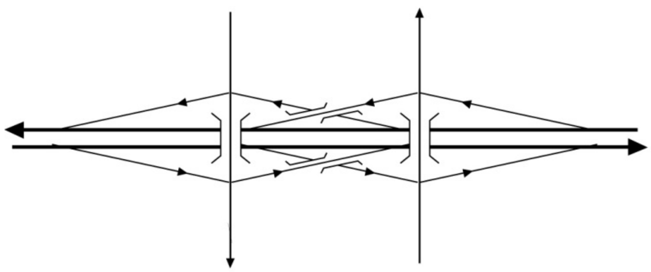
Typical Local Street Interchanges



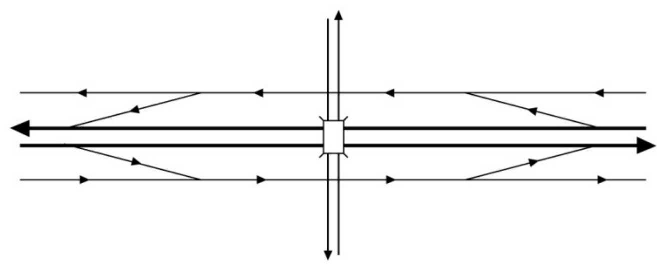
TYPE L-1



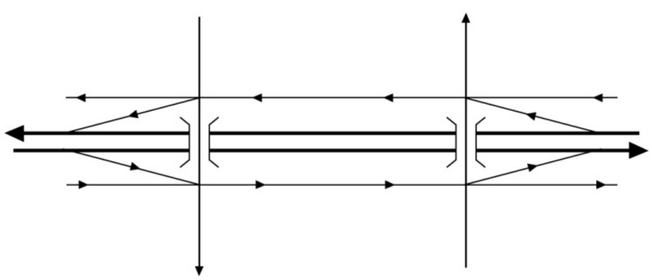
TYPE L-2



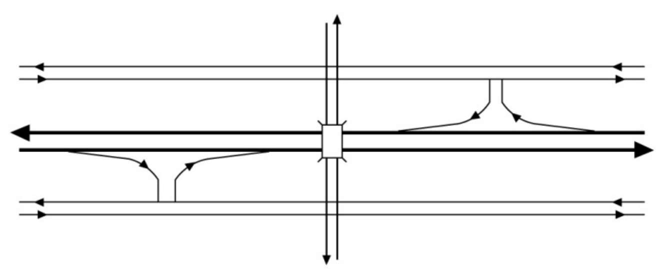
TYPE L-3



TYPE L-4

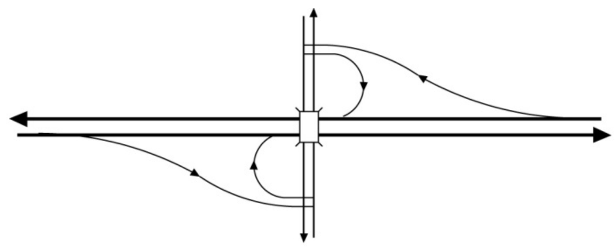


TYPE L-5

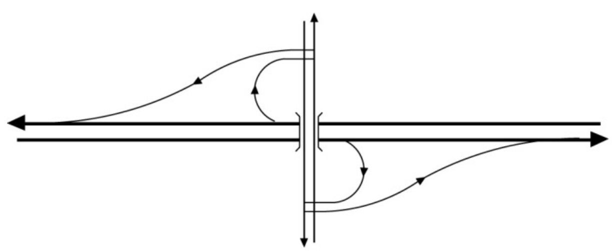


TYPE L-6

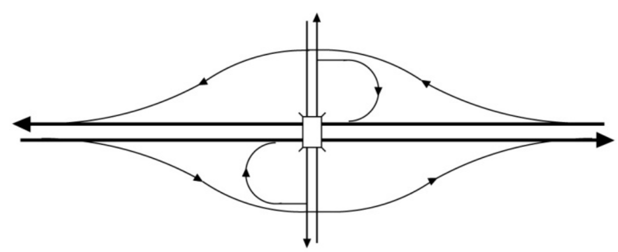
Figure 502.2
Typical Local Street Interchanges
(continued)



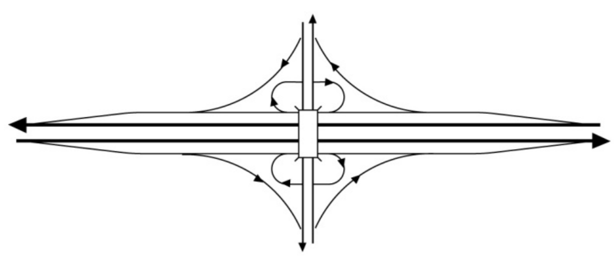
TYPE L-7



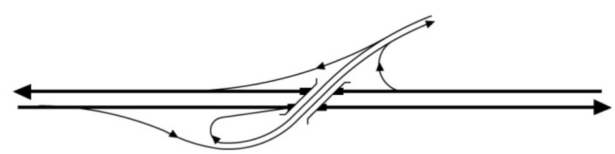
TYPE L-8



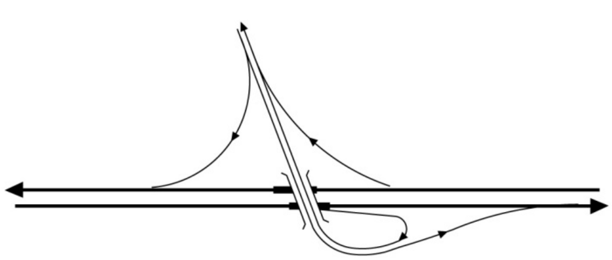
TYPE L-9



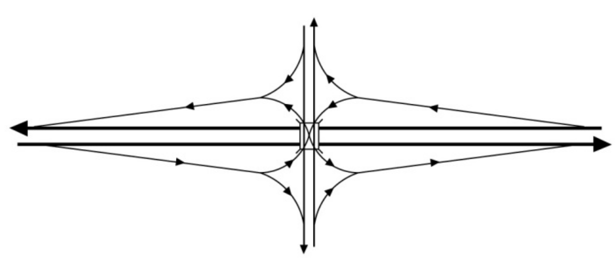
TYPE L-10



TYPE L-11



TYPE L-12



TYPE L-13

elevated and the cross street retains a straight profile. Type L-1's are suitable where physical, geometric or right of way restrictions do not permit a spread diamond configuration. Compact diamonds have the disadvantage of requiring wider overcrossing or longer span undercrossing to provide corner sight distance and have limited capacity between intersections. Once the area around the interchange is developed, Type L-1 is challenging to expand to accommodate growth.

The spread diamond (Type L-2) is adaptable where the grade of the cross street is changed to pass over or under the freeway. The ramp terminals are spread in order to achieve maximum sight distance and minimum intersection cross slope, commensurate with construction and right of way costs, travel distance, and general appearance. A spread diamond has the advantage of flatter ramp grades, greater crossroads left-turn storage capacity, and the flexibility of permitting the construction of future loop ramps if required.

The split diamond with braids (Type L-3) may be appropriate where two major crossroads are closely spaced.

(b) Interchanges with Parallel Street Systems--Types L-4, L-5 and L-6 are interchange systems used where the freeway alignment is placed between parallel streets. Types L-4 and L-5 are used where the parallel streets will operate with one-way traffic. In Type L-4 slip ramps merge with the frontage street and in Type L-5 the ramps terminate at the intersection of the frontage road with the cross street, forming five-legged intersections. In Type L-6 the freeway ramps connect with two-way parallel streets. The parallel streets in the Types L-4, L-5 and L-6 situation are usually too close to the freeway to permit ramp intersections on the cross street between the parallel frontage streets.

The "hook" ramps of the Type L-6 are often forced into tight situations that lead to less than desirable geometrics. The radius of the curve at the approach to the intersection should exceed 150 feet and a tangent of at least 150 feet should be provided between the last curve on the ramp and the ramp terminal. Special attention should always be given to exit ramps that end in a hook to ensure that adequate sight distance around the curve, adequate deceleration length prior to the curve or end of anticipated queue, and adequate superelevation for anticipated driving speeds can be developed. Type L-6 can only be considered when all other interchange types are not acceptable.

(c) Cloverleaf Interchanges--The simplest cloverleaf interchange is the two-quadrant cloverleaf, Type L-7 or Type L-8, or a combination where the two loops are on the same side of the cross street. Type L-7 eliminates the need for left-turn storage lanes, on or under the structure, thus reducing the structure costs. These interchanges should be used only in connection with controls which preclude the use of diamond ramps in all four quadrants. These controls include right of way controls, a railroad track paralleling the cross street, and a short weaving distance to the next interchange.

The Type L-9, partial cloverleaf interchange, provides loop on-ramps in addition to the four diamond-type ramps. This interchange is suitable for large volume turning movements. Left-turn movements from the crossroads are eliminated, thereby permitting two-phase operation at the ramp intersections when signalized. Because of this feature, the Type L-9 interchange usually has capacity to handle the higher volume traffic on the crossroad.

The four-quadrant cloverleaf interchange (Type L-10) offers free-flow characteristics for all movements. It has the disadvantage of a higher cost than a diamond or partial cloverleaf design, as well as a relatively short weaving section between the loop ramps which limits capacity. For this reason this type of interchange is not desirable. Collector-distributor roads should be incorporated in the design of four-quadrant cloverleaf interchanges to separate the weaving conflicts from the through freeway traffic.

(d) Trumpet Interchanges--A trumpet design, Type L-11 or L-12, may be used when a crossroads terminates at a freeway. This design should not be used if future extension of the crossroads is probable. The diamond interchange is preferable if future extension of the crossroads is expected.

(e) Single Point Interchange (SPI)--The Type L-13 is a concept which essentially combines two separate diamond ramp intersections into one large at-grade intersection. It is also known as an urban interchange. Additional information on SPI's is provided in the Single Point Interchange Planning, Design and Operational Guidelines (SPI Guidelines), issued by memorandum on June 15, 2001.

Type L-13 requires approximately the same right of way as the compact diamond. However, the construction cost is substantially higher due to the structure requirements. The capacity of the L-13 can exceed that of a compact diamond if long signal times can be provided and left turning volumes are balanced.

This additional capacity may be offset if nearby intersection queues interfere with weaving and storage between intersections. The disadvantages of the L-13 are: 1) future expansion of the interchange is extremely difficult; 2) stage construction for retrofit situations is costly; 3) long structure spans require higher than normal profiles and deeper structure depths; and 4) poor bicycle and pedestrian circulation.

(f) Other Types of Interchanges--New or experimental interchanges must have the Project Delivery Coordinator and Headquarters Traffic Liaison's concurrence before selection. Concurrence may require additional studies and documentation.

502.3 Freeway-to-Freeway Interchanges

(1) General. The function of the freeway-to-freeway interchange is to link freeway segments together so as to provide the highest level of service in terms of mobility. Parameters such as cost, environment, community values, traffic volumes, route continuity, driver expectation and safety should all be considered. Route continuity, providing for the designated route to continue as the through movement through an interchange, reduces lane changes, simplifies signing, and reduces driver confusion.

Interstate routes shall maintain route continuity. Where both the designated route and heavier traffic volume route are present, the interchange configuration shall keep the designated route to the left through the interchange.

(2) Design Considerations.

(a) Cost--The differential cost between interchange types is often significant. A cost-effective approach will tend to assure that an interchange is neither over nor underdesigned. Decisions as to the relative values of the previously mentioned parameters must be consistent with decisions reached on adjacent main line freeways.

(b) System Balance--The freeway-to-freeway interchange is a critical link in the total freeway system. The level of traffic service provided will have impact upon the mobility and overall effectiveness of the entire roadway system. For instance, traffic patterns will adjust to avoid repetitive bottlenecks, and to the greatest degree possible, to temporary closures, accidents, etc. The freeway-to-freeway interchange should provide flexibility to respond to these needs so as to maximize the cost effectiveness of the total system.

(c) Provide for all Traffic Movements--All interchanges must provide for each of the eight basic movements (or four basic movements in the case of a three-legged interchange), except in the most extreme circumstances. Less than "full interchanges" may be considered on a case-by-case basis for applications requiring special access for managed lanes (e.g., transit, HOVs, HOT lanes) or park and ride lots. Partial interchanges usually have undesirable operational characteristics. If circumstances exist where a partial interchange is considered appropriate as an initial phase improvement, then commitments need to be included in the request to accommodate the ultimate design. These commitments may include purchasing the right of way required during the initial phase improvements.

(d) Local Traffic Service--In metropolitan areas a freeway-to-freeway interchange is usually superimposed over an existing street system. Local and through traffic requirements are often in conflict.

Combinations of local and freeway-to-freeway interchanges can result in designs that are both costly and so complex that the important design concepts of simplicity and consistency are compromised. Therefore, alternate plans separating local and freeway-to-freeway interchanges should be fully explored. Less than desirable local interchange spacing may result; however, this may be compensated for by upgrading the adjacent local interchanges and street system.

Local traffic service interchanges should not be located within freeway-to-freeway interchanges unless geometric standards and level of service will be substantially maintained.

(e) Alignment--It is not considered practical to establish fixed freeway-to-freeway interchange alignment standards. An interchange must be designed to fit into its environment. Alignment is often controlled by external factors such as terrain, buildings, street patterns, route adoptions, and community value considerations. Normally, loops have radii in the range of 150 feet to 200 feet and direct connections should have minimum radii of 850 feet. Larger radii may be proper in situations where the skew or other site conditions will result in minimal increased costs. Direct connection radii of at least 1,150 feet are desirable from a traffic operational standpoint. High alignment and sight distance standards should be provided where possible.

Drivers have been conditioned to expect a certain standard of excellence on California freeways. The designer's challenge is to provide the highest possible standards consistent with cost and level of service.

(3) Types. Several freeway-to-freeway interchange design configurations are shown on Figure 502.3. Many combinations and variations may be formed from these basic interchange types.

(a) Four-Level-Interchange--Direct connections are appropriate in lieu of loops when required by traffic demands or other specific site conditions. The Type F-1 interchange with all direct connections provides the maximum in mobility and safety. However, the high costs associated with this design require that the benefits be fully substantiated.

The Type F-1 Alternative "A" interchange utilizes a single divergence ramp for traffic bound for the other freeway; then provides a secondary directional split. Each entrance ramp on a Type F-1A interchange is provided separately. The advantages of the Type F-1A are: 1) reduced driver confusion since there is only one exit to the other freeway, and 2) operations at the entrance may be improved since the ramps merge with the mainline one at a time.

The Type F-1 Alternative "B" interchange provides separate directional exit ramps and then merges the entering traffic into a single ramp before converging with the mainline. Since the Type F-1B combines traffic from two ramps before entering the freeway, it is important to verify that adequate weaving capacity is provided beyond the entrance. Separating the directional split of exiting traffic reduces the volume to each of the two ramps and therefore may improve the level of service of the weave section prior to the exit.

Design for a four-level interchange may combine the configuration of the Type F1-A and F1-B interchange to best suit the conditions at a given location.

(b) Combination Interchanges--The three-quadrant cloverleaf, Type F-2, with one direct connection may be necessary where a single move carries too much traffic for a loop ramp or where the one quadrant is restricted by environmental, topographic, or right of way controls.

The two-loop, two-direct connection interchange, Type F-3, is often an appropriate solution. The weaving conflicts which ordinarily constitute the most restrictive traffic constraint are eliminated, yet cost and right of way requirements may be kept within reasonable bounds. Consideration should be given to providing an auxiliary lane in advance of the loop off-ramps to provide for vehicle deceleration.

(c) Four-Quadrant Cloverleaf--The four-quadrant cloverleaf with collector-distributor roads, Type F-4, is ordinarily the most economical freeway-to-freeway interchange solution when all turning movements are provided. The four-quadrant cloverleaf is generally applicable in situations where turning volumes are low enough to be accommodated in the short weaving sections. It should be designed with collector-distributor roads to separate weaving conflicts from the through freeway traffic.

(d) Freeway Terminal Junction--Types F-5, F-6, F-7, and F-8 are examples of interchange designs where one freeway terminates at the junction with another freeway. In general, the standard of alignment provided on the left or median lane connection from the terminating freeway should equal or approach as near as possible that of the terminating freeway. Terminating the median lane on a loop should be avoided. It is preferable that both the designated route and the major traffic volume be to the left at the branch connection diverge. The choice between Types F-7 and F-8 should include considerations of traffic volumes, and route continuity. When these considerations are in conflict, the choice is made on the basis of judgment of their relative merits.

Topic 503 - Interchange Design Procedure

503.1 Basic Data

Data relative to community service, traffic, physical and economic factors, and potential area development which may materially affect design, should be obtained prior to interchange design. Specifically, the following information should be available:

(a) The location and standards of existing and proposed local streets including types of traffic control.

(b) Existing, proposed and potential for development of land, including such developments as employment centers, retail services and shopping centers, recreational facilities, housing developments, schools, and other institutions.

(c) A vehicle traffic flow diagram showing average daily traffic and design hourly volumes, as well as time of day (a.m. or p.m.), anticipated on the freeway ramps and affected local streets or roads.

(d) Current and future bicycle and pedestrian access through the community.

(e) The relationship with adjacent interchanges.

(f) The location of major utilities, railroads, or airports.

(g) The presence of dedicated lanes and associated ramps and connections, including HOV lanes, Bus (BRT) lanes and Express lanes.

(h) The planned ultimate build-out for the freeway facility.

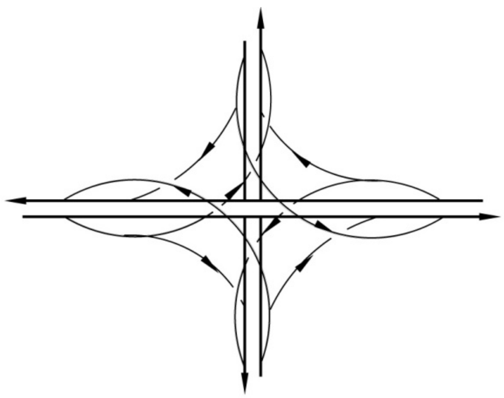
(i) Existing and planned rail facilities.

503.2 Reviews

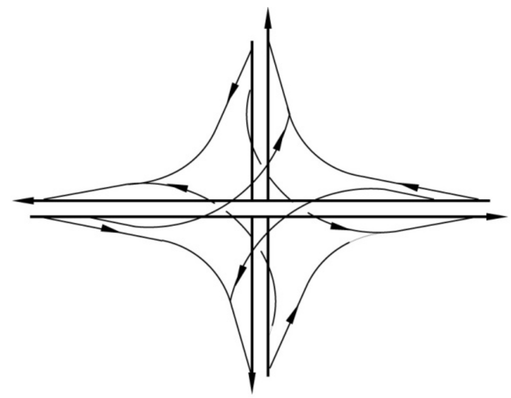
Interchanges are among the major design features which are to be reviewed by the Project Delivery Coordinator and/or District Design Liaison, HQ Traffic Liaison, other Headquarters staff, and the FHWA Transportation Engineer, as appropriate. Major design features include the freeway alignment, geometric cross

Figure 502.3

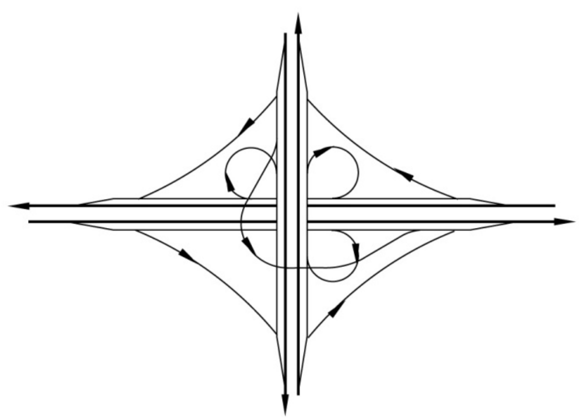
Typical Freeway-to-freeway Interchanges



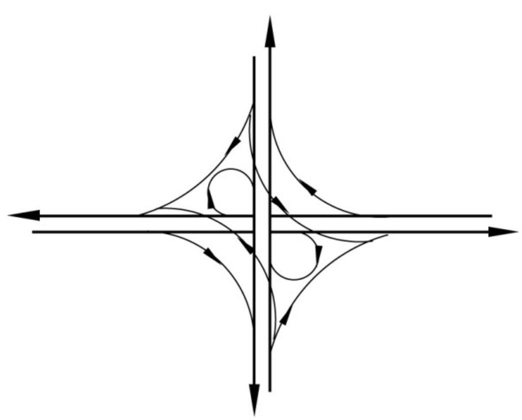
TYPE F-1 (ALT "A")



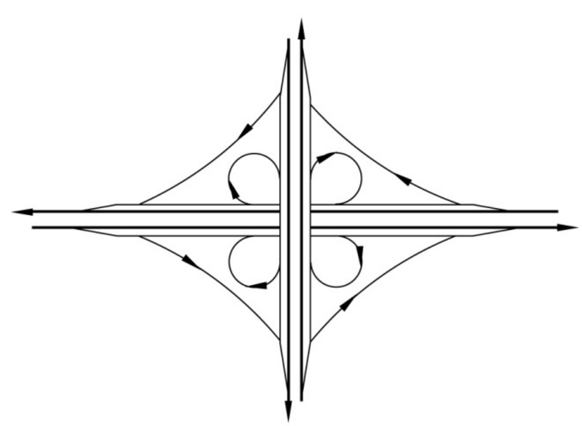
TYPE F-1 (ALT "B")



TYPE F-2



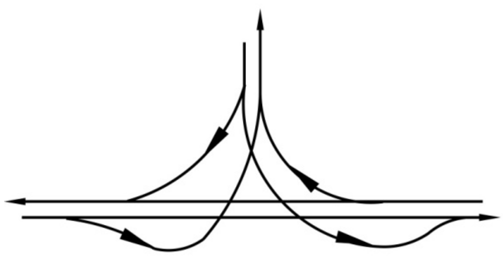
TYPE F-3



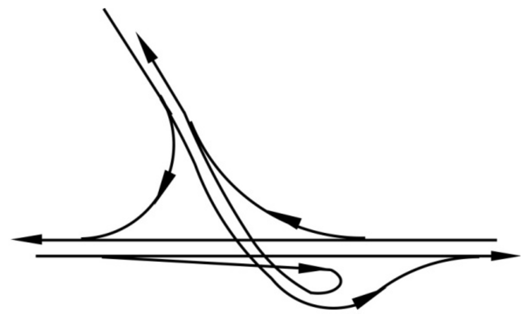
TYPE F-4

Figure 502.3

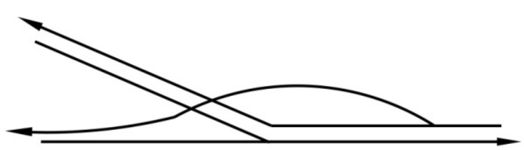
Typical Freeway-to-freeway Interchanges (continued)



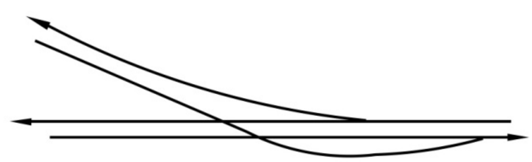
TYPE F-5



TYPE F-6



TYPE F-7



TYPE F-8

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 Project Delivery 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.* <u>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.</u>

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. <u>The same standard should apply for the first curve after the exit from a collector-distributor road.</u> 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).

<u>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.</u> Also, see pavement cross slope requirements in Index 504.2(5).

<u>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.</u> 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

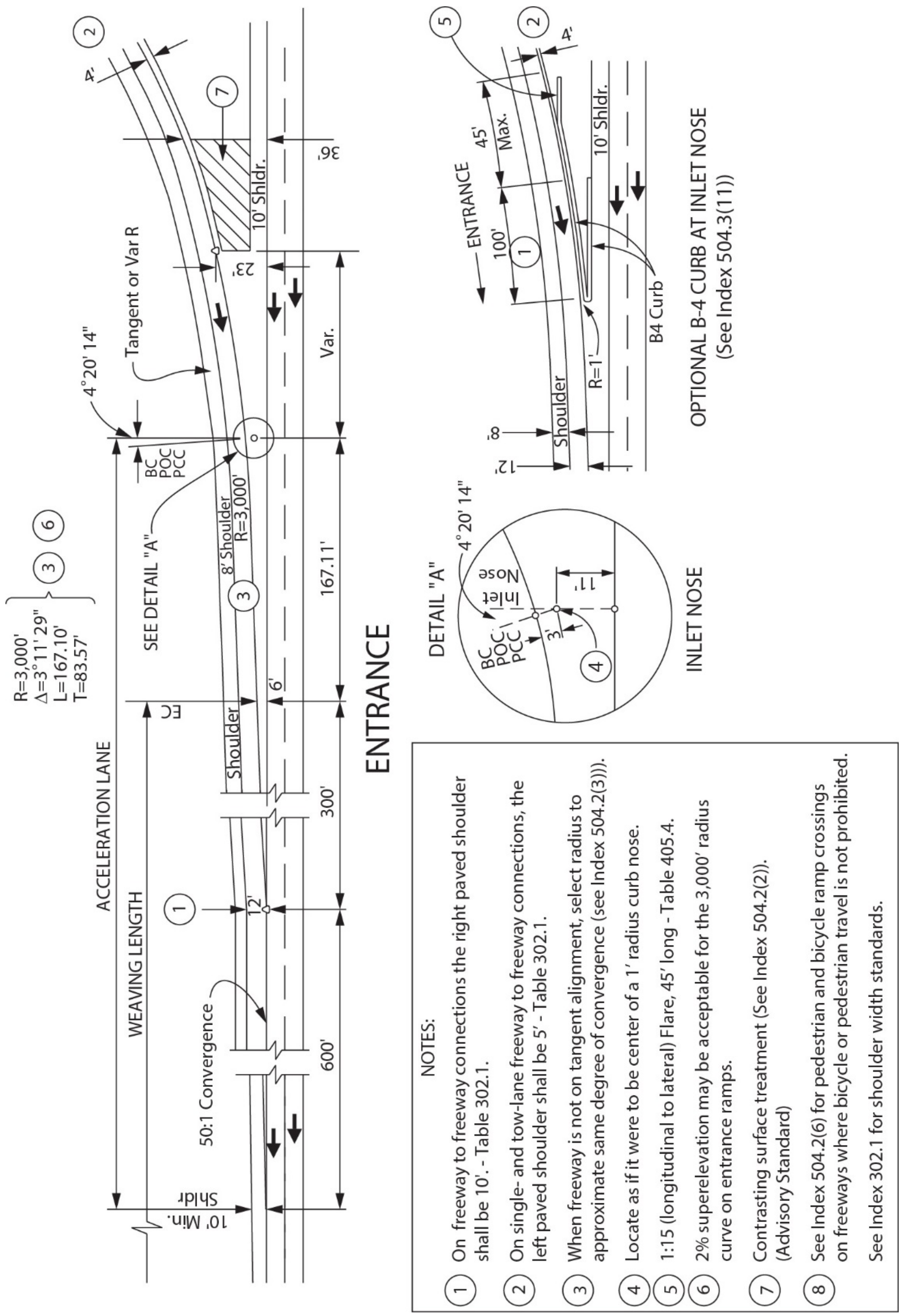
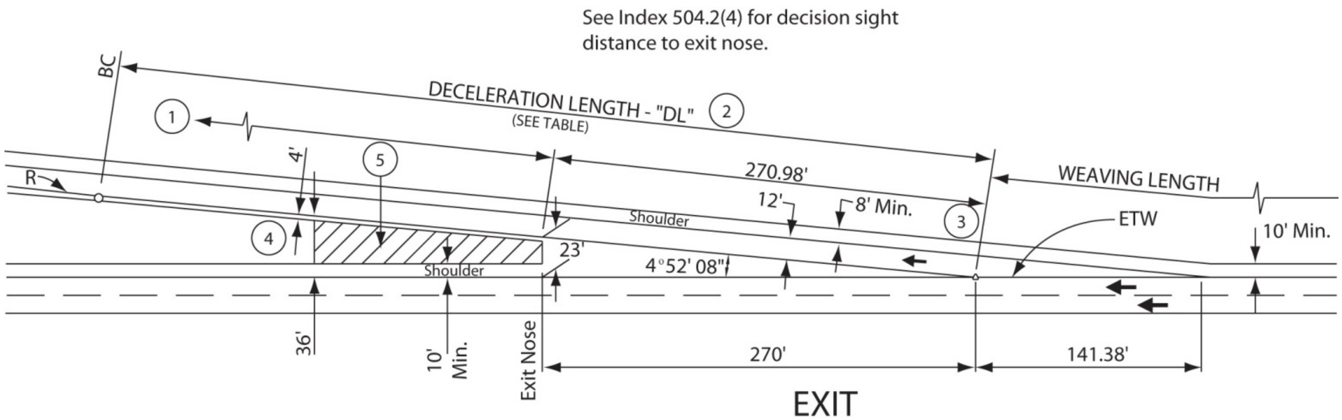


Figure 504.2B

Single Lane Freeway Exit



R (ft)	Min. DL (ft) 2
Less than 300	570
300 - 499	470
500 - 999	420
1,000 & over	270

NOTES:

1. Minimum length between exit nose and end of ramp is 525' for full stop at end of ramp.
2. "DL" distance should be lengthened for descending, short radius curves, or if entered from a sustained downgrade.
3. On freeway to freeway connections the right paved shoulder shall be 10'. - Table 302.1
4. On single- and two-lane freeway to freeway connections the left paved shoulder shall be 5'. - Table 302.1
5. Contrasting surface treatment (See Index 504.2(2)) (Advisory standard)

See Index 302.1 for shoulder width standards.

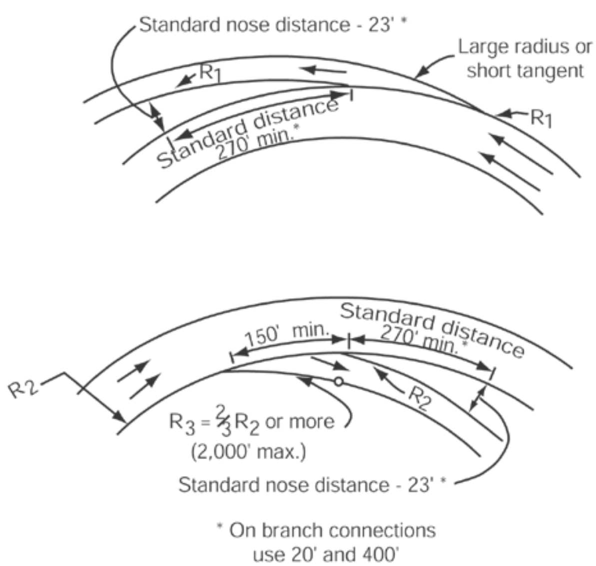
On entrance ramps the distance from the inlet nose (14-foot point) to the end of the acceleration lane taper should equal the sum of the distances shown on Figure 504.2A. The 50:1 (longitudinal to lateral) taper may be curved to fit the conditions, and the 3,000-foot radius curve may be adjusted (see Figure 504.2A, note 3).

When an exit must be located where physical restrictions to visibility cannot be corrected by cut widening or object removal, an auxiliary lane in advance of the exit should be provided. The length of auxiliary lane should be a minimum 600 feet, 1,000 feet preferred.

(4) *Design Speed Considerations.* In the design of interchanges it is important to provide vertical and horizontal alignment standards which are consistent with driving conditions expected on branch connections. Sight distance on crest vertical curves should be consistent with expected approach speeds.

(a) Freeway Exit--The design speed at the exit nose should be 50 miles per hour or greater for both ramps and branch connections.

Figure 504.2C
Location of Freeway Ramps on a Curve



Decision sight distance given in Table 201.7 should be provided at freeway exits and branch connectors. At secondary exits on collector-distributor roads, a minimum of 600 feet of decision sight distance should be provided. In all cases, sight distance is measured to the center of ramp lane right of the nose.

(b) Freeway Entrance--The design speed at the inlet nose should be consistent with approach alignment standards. If the approach is a branch connection or diamond ramp with high alignment standards, the design speed should be at least 50 miles per hour.

(c) Ramps--See Index 504.3(1)(a).

(d) Freeway-to-Freeway Connections--See Index 504.4(2).

(5) *Grades.* Grades for freeway entrances and exits are controlled primarily by the requirements of sight distance. Ramp profile grades should not exceed 8 percent with the exception of descending entrance ramps and ascending exit ramps, where a 1 percent steeper grade is allowed. However, the 1 percent steeper grade should be avoided on descending loops to minimize overdriving of the ramp (see Index 504.3 (8)).

Profile grade considerations are of particular concern through entrance and exit gore areas. In some instances the profile of the ramp or connector, or a combination of profile and cross slope, is sufficiently different than that of the freeway through lanes that grade breaks across the gore may become necessary. Where adjacent lanes or lanes and paved gore areas at freeway entrances and exits are not in the same plane, the algebraic difference in pavement cross slope should not exceed 5 percent (see Index 301.2).The paved gore area is typically that area between the diverging or converging edge of traveled ways and the 23-foot point.

In addition to the effects of terrain, grade lines are also controlled by structure clearances (see Indexes 204.6 and 309.2). Grade lines for overcrossing and undercrossing roadways should conform to the requirements of HDM Topic 104 Roads Under Other Jurisdictions.

(a) Freeway Exits--Vertical curves located just beyond the exit nose should be designed with a minimum 50 miles per hour stopping sight distance. Beyond this point, progressively lower design speeds may be used to accommodate loop ramps and other geometric features.

Ascending off-ramps should join the crossroads on a reasonably flat grade to expedite truck starts from a stopped condition. If the ramp ends in a crest vertical curve, the last 50 feet of the ramp should be on a 5 percent grade or less. There may be cases where a drainage feature is necessary to prevent crossroads water from draining onto the ramp.

On descending off-ramps, the sag vertical curve at the ramp terminal should be a minimum of 100 feet in length.

(b) Freeway Entrances--Entrance profiles should approximately parallel the profile of the freeway for at least 100 feet prior to the inlet nose to provide intervisibility in merging situations. The vertical curve at the inlet nose should be consistent with approach alignment standards.

Where truck volumes (three-axle or more) exceed 20 per hour on ascending entrance ramps to freeways and expressways with sustained upgrades exceeding 2 percent, a 1,500-foot length of auxiliary lane should be provided in order to ensure satisfactory operating conditions. Additional length may be warranted based on the thorough analysis of the site specific grades, traffic volumes, and calculated speeds; and after consultation with the HQ Traffic Liaison and the Project Delivery Coordinator or District Design Liaison. Also, see Index 204.5 "Sustained Grades".

(6) *Bus Stops.* See Index 108.2 and 303.4 for general information.

(7) *Bicycle and Pedestrian Conditions*. On freeways where bicycle or pedestrian travel is not prohibited, provisions need to be made at interchanges to accommodate bicyclists and pedestrians. See Topic 116 and the California MUTCD for additional guidance.

504.3 Ramps

(1) *General.*

(a) Design Speed--When ramps terminate at an intersection at which all traffic is expected to make a turning movement, the minimum design speed along the ramp should be 25 miles per hour. When a "through" movement is provided at the ramp terminus, the minimum ramp design speed should meet or exceed the design speed of the highway facility for which the through movement is provided. The design speed along the ramp will vary depending on alignment and controls at each end of the ramp. An acceptable approach is to set design speeds of 25 miles per hour and 50 miles per hour at the ramp terminus and exit nose, respectively, the appropriate design speed for any intermediate point on the ramp is then based on its location relative to those two points. When short radius curves with relatively lower design speeds are used, the vertical sight distance should be consistent with approach vehicle speeds. See Index 504.2(4) for additional information regarding design speed for ramps.

(b) Lane Width--**Ramp lanes shall be a minimum of 12 feet in width. Where ramps have curve radii of 300 feet or less, measured along the outside edge of traveled way for single lane ramps or along the outside lane line for multilane ramps, with a central angle greater than 60 degrees, the single ramp lane, or the lane furthest to the right if the ramp is multilane, shall be widened in accordance with Table 504.3 in order to accommodate large truck wheel paths.** See Topic 404. Consideration may be given to widening more than one lane on a multilane ramp with short radius curves if there is a likelihood of considerable transit or truck usage of that lane.

Table 504.3
Ramp Widening for Trucks

Ramp Radius (ft)	Widening (ft)	Lane Width (ft)
<150	6	18
150 – 179	4	16
180 – 209	3	15
210 – 249	2	14
250 – 299	1	13
>300	0	12

(c) Shoulder Width--**Shoulder widths for ramps shall be as indicated in Table 302.1.** Typical ramp shoulder widths are 4 feet on the left and 8 feet on the right.

(d) Lane Drops--Typically, lane drops are to be accomplished over a distance equal to WV. Where ramps are metered, the recommended lane drop taper past the meter limit line is 50 to 1 (longitudinal to lateral). Depending on approach geometry and speed, the lane drop transition between the limit line and the 6-foot separation point should be accomplished with a taper of between 30:1 and 50:1 (longitudinal to lateral). This is further explained in Index 504.3(2)(b) for metered multilane entrance ramps. **However, the lane drop taper past the limit line shall not be less than 15 to 1.**

Lane drop tapers should not extend beyond the 6-foot point (the beginning of the weaving length) without the provision of an auxiliary lane.

(e) Lane Additions -- Lane additions to ramps are usually accomplished by use of a 120-foot bay taper. See Table 405.2A for the geometrics of bay tapers.

(2) Ramp Metering

All geometric designs for ramp metering installations must be discussed with the Project Delivery Coordinator or District Design Liaison. Design features or elements which deviate from the mandatory standards require the approvals described in Index 82.2. Before beginning any ramp meter design, the designer must contact District Traffic Operations for direction in the application of procedural requirements of the Division of Traffic Operations.

Geometric ramp design for operational improvement projects for ramp meters should be based on current peak-hour traffic volume (this is considered to be data that is less than two years old). If this data is not available it should be obtained before proceeding with design. Peak hour traffic data from the annual Traffic Volumes book is not adequate for this application.

The design advice and typical designs that follow should not be directly applied to ramp meter installation projects, especially retrofit designs, without giving consideration to "customizing" the geometric design features to meet site and traffic conditions (i.e., design highway volume, geometry, speeds, etc.). Every effort should be made by the designer to exceed the recommended minimum standards provided herein, where conditions are not restrictive.

(a) Metered Single-Lane Entrance Ramps

Geometrics for a single-lane ramp meter should be provided for volumes up to 900 vehicles per hour (vph) (see Figures 504.3A and 504.3B). Where truck volumes (3-axle or more) are 5 percent or greater on ascending entrance ramps to freeways with sustained upgrades exceeding 3 percent (i.e., at least throughout the merge area), a minimum 500-foot length of auxiliary lane should be provided beyond the ramp convergence point. For additional guidance see AASHTO, A Policy on Geometric Design of Highways and Streets.

A multilane ramp segment may be provided to increase vehicle storage within the available ramp length (see Index 504.3(2)(d) Storage Length) and/or to create a preferential lane for High-Occupancy Vehicles (HOV)s, as required in Index 504.3(2)(h).

(b) Metered Multilane Ramps

When entrance ramp volumes exceed 900 vph, and/or when a HOV preferential lane is provided, a two- or three-lane ramp segment should be provided. Figures 504.3C, 504.3D and 504.3E illustrate typical designs for metered two-lane ramps; and Figures 504.3F and 504.3G illustrate typical designs for metered three-lane ramps. On two-lane loop ramps, normally only the right lane needs to be widened to accommodate design vehicle off-tracking. See Index 504.3(1)(b).

Three-lane metered ramps are typically needed to serve peak (i.e., commute) hour traffic along urban and suburban freeway corridors. The adverse effects of bus and truck traffic on the operation of these ramps (i.e., off-tracking, sight restriction, acceleration characteristics on upgrades, etc.) is minimized when the ramp alignment is tangential or consists of curve radii not less 300 feet.

On local street entrance ramps, the multi-lane segment should transition to a single lane width between the ramp meter limit line and the 6-foot separation point (from the mainline edge of traveled way). See Figures 504.3C, 504.3D, 504.3E, 504.3F, 504.3G, 504.3H and 504.3I.

The lane drop transition should be accomplished with a taper of 50:1 (longitudinal to lateral) unless a lesser taper is warranted by site and/or project specific conditions which control the ramp geometry and/or anticipated maximum speed of ramp traffic. For example, "loop" entrance ramps would normally not allow traffic to attain speeds which would warrant a 50:1 (longitudinal to lateral) lane drop taper. Also, in retrofit situations, existing physical, environmental or right of way constraints may make it impractical to provide a 50:1 taper, especially if the maximum anticipated approach speed will be less than 50 miles per hour. Therefore, depending on approach geometry and speed, the lane drop transition between the limit line and the 6-foot separation point should be accomplished with a taper of between 30:1 and 50:1 (longitudinal to lateral). **However, the lane drop taper past the limit line shall not be less than 15 to 1.**

Where truck volumes (3-axle or more) are 5 percent or greater on ascending entrance ramps to freeways with sustained upgrades exceeding 3 percent (i.e. at least throughout the merge area), a minimum 1,000 feet length of auxiliary lane should be provided beyond the ramp convergence point. AASHTO, A Policy on Geometric Design of Highways and Streets, provides additional guidance on acceleration lane length on grades.

When ramp volumes exceed 1,500 vph, a 1,000-foot minimum length of auxiliary lane should be provided beyond the ramp convergence point. If an auxiliary lane is included, the ramp lane transition may be extended to the convergence point. However, the proximity of the nearest interchange may warrant weaving analysis to determine the acceptability of extending the ramp lane transition beyond the 6-foot separation point. A longer auxiliary lane should be considered where mainline/ramp gradients and truck volumes warrant additional length.

(c) Metered Freeway-to-Freeway Connectors

Freeway-to-freeway connectors may also be metered when warranted. The need to meter a freeway-to-freeway connector should be determined on an individual basis. Because connector ramps provide a link between two high speed facilities, drivers do not expect to stop, nor do they expect to approach a stopped vehicle.

The installation of ramp meters on connector ramps shall be limited to those facilities which meet or exceed the following geometric design criteria:

- **Standard lane and shoulder widths.**
- **"Tail light" sight distance, measured from 3 ½ feet eye height to a 2-foot**

object height, is provided for a design speed of 50 miles per hour minimum.

All lane drop transitions on connectors shall be accomplished with a taper of 50:1 (longitudinal to lateral) minimum, (see Figures 504.3H and 504.3I).

(d) Storage Length

In keeping the Strategic Plan to maximize the effectiveness of operational strategies, an important design consideration for a ramp meter system is providing adequate storage for queues. The District Operations Branch responsible for ramp metering must be consulted to determine the desirable ramp meter storage.

Ramp meters have practical lower and upper output limits of 240 and 900 vph per lane, respectively. Ramp meter signals set for flow rates outside this range tend to have high violation rates and cannot effectively control traffic. Therefore, on a ramp with peak hour volume between 500 and 900, a two-lane ramp meter may be provided to double the vehicles stored within the available storage area. A single-lane ramp meter should be used when rates are below 500 vph and no HOV preferential lane is provided.

To minimize the impact on local street operation, every effort should be made to meet the recommended storage length. Wherever feasible, ramp metering storage should be contained on the ramp by either widening or lengthening it. Improvements to the local street system in the vicinity of the ramp should also be thoroughly investigated where there is insufficient storage length on the ramp and the ramp queue will adversely affect local street operation. Note that excessive queue length may also impact the mobility of pedestrians and bicyclists. The storage length that can be provided on the ramp may be limited by the weaving distance to the next off-ramp and/or available right of way. Local street improvements can include widening or restriping the street(s) or intersection(s) to provide additional storage or capacity. Signal timing revisions along the corridor feeding the ramp can also enhance the storage capability. These will require coordination with the local agency consistent with the regional traffic operations strategy. Ultimately system-wide adaptive ramp metering will coordinate with local street and arterial signal systems.

The current peak period 5, 6, or 15 minute arrival rates and anticipated or current ramp meter discharge rates should be used to determine the storage length required for ramp metering. It is recommended that a minimum vehicle spacing of 30 feet be used for designing storage on metered ramps. Additional spacing should be provided for locations where there are significant percentages of trucks, buses or recreational vehicles.

It is the responsibility of the Department, on Department initiated projects, to mitigate the effect of ramp metering, for initial as well as future operational impacts, on local streets that intersect and feed entrance ramps to the freeway. Developers and/or local agencies, however, should be required to mitigate any impact to existing ramp meter facilities, future ramp meter installations, or local streets, when those impacts are attributable to new development and/or local agency roadway improvement projects.

(e) Pavement Structure

In planning for the possibility of future widening, the pavement structure for the ramp shoulders should be equal to the ramp traveled way pavement structure. In locations where failure of loop detectors due to flexible pavement deterioration is a concern, a Portland Cement Concrete (PCC) pad may be considered on new construction and rehabilitation projects. The concrete pad should cover the metering detector loop area upstream and downstream of the limit line.

(f) Meter Location

On single-lane ramps, the ramp meter signal standard should be placed on the driver's left.

(g) Limit Line Location

The limit line location will be determined by the selected transition taper, but should be a minimum of 75 feet upstream of the 23-foot point on the entrance ramp. A single 12-inch solid white line will be placed across all metered lanes. Staggered limit lines should not be used.

(h) HOV Preferential Lane

Ramp meter installations should operate in conjunction with, and complement other transportation management system elements and transportation modes. As such, ramp meter installations should include preferential treatment of carpools and transit riders. Specific treatment(s) must be tailored to the unique conditions at each ramp location, however the standard or base treatment upon which other strategies are designed is the High-Occupancy Vehicle (HOV) preferential lane.

Division of Traffic Operations policy requires an HOV preferential lane be provided at all ramp meter locations. Deviation from this policy requires concurrence from the Headquarters (HQ) Traffic Liaison, which must be reflected in the Project Initiation Document.

In general, the vehicle occupancy requirement for ramp meter HOV preferential lanes will be two or more persons per vehicle. At some locations, a higher vehicle occupancy requirement may be necessary. The occupancy should be based on the HOV demand and coordination with other HOV facilities in the vicinity.

A HOV preferential lane should typically be placed on the left; however, demand and operational characteristics at the ramp entrance may dictate otherwise. The District Operations Branch responsible for ramp metering will determine which side of the ramp the HOV preferential lane will be placed, and whether or not it will be metered.

- It is the policy of Districts 4, 6, 8, and 11 to meter the HOV preferential lane.
- Districts 3, 7, and 12 typically do not meter the HOV preferential lane.

Access to the HOV preferential lane may be provided in a variety of ways depending on interchange type and the adequacy of storage provided for queued vehicles. Where queued vehicles are expected to block access to the HOV preferential lane, direct or separate access should be considered. Designs should consider pedestrian/bicycle volumes, especially when the entrance ramp is located near a school or the local highway facility includes a designated bicycle lane or route. See Index 403.6 for requirement for turn-only lanes. Contact the HQ Traffic Liaison and the Project Delivery Coordinator or District Design Liaison to discuss the application of specific design and/or general issues related to the design of HOV preferential lane access.

Signing for a HOV preferential lane should be placed to clearly indicate which lane is designated for HOVs. Real-time signing at the ramp entrance, such as an overhead changeable message sign, may be necessary at some locations if pavement delineation and normal signing do not provide drivers with adequate lane usage information. To avoid leading Single-Occupancy Vehicles (SOV) into a HOV preferential lane, pavement delineation at the ramp entrance should lead drivers into the SOV lane.

(i) Modifications to Existing HOV Preferential Lanes

Changes in traffic conditions, proposals for interchange modifications, recurrent operational problems affecting the local facility, or the need to further improve mainline operations through more restrictive metering all provide an opportunity to reevaluate the need for a HOV preferential lane. HOV preferential lanes should remain in place or be added to the scope of projects generated in response to any of the above scenarios. Alternate solutions should be investigated before removal is considered. For example: better control over ramp traffic can be attained by retrofitting ramps to meter HOV traffic which bypasses the ramp meter. Underutilization of an existing lane plus the need for additional right of way for storage, the availability of an alternate HOV entrance ramp within 1½ mile, or the availability of a direct HOV access (drop) ramp will typically provide adequate justification for the removal of a HOV preferential lane at specific locations.

The Deputy District Director of Operations, in consultation with the HQ Traffic Liaison, is responsible for approving decisions to remove HOV preferential lanes. Written documentation should be provided in the appropriate project document(s).

(j) Enforcement Areas and Maintenance Pullouts

Division of Traffic Operations policy requires an enforcement area be provided on all two-lane and three-lane on-ramps with HOV preferential lanes. Deviation from this policy requires concurrence from the HQ Traffic Liaison, which must be reflected in the Project Initiation Document.

On single-lane ramps, a paved enforcement area is not necessary, but the area should be graded to facilitate future ramp widening (see Figure 504.3A). Enforcement areas are used by the California Highway Patrol (CHP) to enforce minimum vehicle occupancy requirements. At locations where the HOV preferential lane is metered, the CHP enforcement area should begin as close to the limit line as practical. Where unmetered, it should begin approximately 170 feet downstream of the limit line. On three-lane ramps, the CHP enforcement area should be downstream of the mast arm standard, approximately 70 feet from the limit line. The length of the CHP enforcement area and its distance downstream of the limit line may be adjusted to fit conditions at the ramp with CHP approval.

The District Traffic Operations Branch responsible for ramp metering must coordinate enforcement issues with the CHP. The CHP Area Commander must be contacted during the Project Report stage, prior to design, to discuss any variations needed to the CHP enforcement area designs shown in this manual. Variations must be discussed with the HQ Traffic Liaison and the Project Delivery Coordinator and/or District Design Liaison.

A paved pullout area near the controller cabinet should be provided for safe and convenient access for Maintenance and Operations personnel. If a pullout cannot be provided, a paved or "all weather" walkway should be provided to the controller cabinet, see Index 107.2. See Topic 309, Clearances, for placement guidance of fixed objects such as controller cabinets.

(3) Location and Design of Ramp Intersections on the Crossroads.

Factors which influence the location of ramp intersections on the crossroads include sight distance, construction and right of way costs, bicycle and pedestrian mobility, circuitous travel for left-turn movements, crossroads gradient at ramp intersections, storage requirements for left-turn movements off the crossroads, and the proximity of other local road or bicycle path intersections.

Ramp intersections with local roads are intersections at grade. Chapter 400 and the references therein contain general guidance. For ramp intersections, a wrong-way movement onto an off-ramp can have severe consequences. The California MUTCD also contains guidance for signing and striping to deter wrong-way movements.

Interchange Types L-7, L-8, and L-9 are partial cloverleaf designs with ramps at a right angle to the crossroad where the off-ramps and on-ramps are adjacent to each other on the same side of the crossroad that offer benefits for non-motorized travel modes; however, additional design considerations as follows may be appropriate in order to deter wrong-way movements:

- The entrance and exit ramps should be clearly visible from the crossroad. Concrete barrier or metal beam guardrail placed between the ramps can block the view from the crossroad. If feasible, the concrete barrier or metal beam guardrail channelization feature should be set back from the crossroad edge of shoulder 20 to 50 feet with a raised traffic island placed from the ramp termini to the begin point of the separation feature. See Index 405.4 for further traffic island guidance. Consult the District Traffic Safety Branch for available options.
- Vehicles turning left onto an on-ramp are to be prevented, to the maximum extent feasible, from turning prematurely onto the off-ramp by placing or extending a curbed median on the crossroad to physically discourage this move. Attention needs to be given to accommodating truck turn templates for design vehicles entering and exiting the freeway. See Index 404.5 for further turning template guidance. Truck aprons could be provided if the size of an intersections becomes too large for an occasional truck. See Index 405.10, Roundabouts, and the references therein for design guidance on truck aprons.

Isolated off-ramps are to be avoided to minimize the potential for wrong-way movements. If the isolated off-ramp is necessary, the leading curb return from the perspective of a vehicle on the crossroad approaching from the same side as the off-ramp is made with a short radius curve of 3 to 5 feet. State or local roads and driveways opposite isolated off-ramps are to be avoided as there is no corresponding on-ramp for cross traffic to take. See this chapter for further interchange and ramp guidance.

<u>Ramp terminals should connect where the grade of the overcrossing is 4 percent or less to avoid potential overturning of trucks.</u>

<u>For left-turn maneuvers from an off-ramp at an unsignalized intersection, the length of 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.</u>

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 Project Delivery Coordinator or the District Design Liaison and then documented as required by the Project Delivery 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.

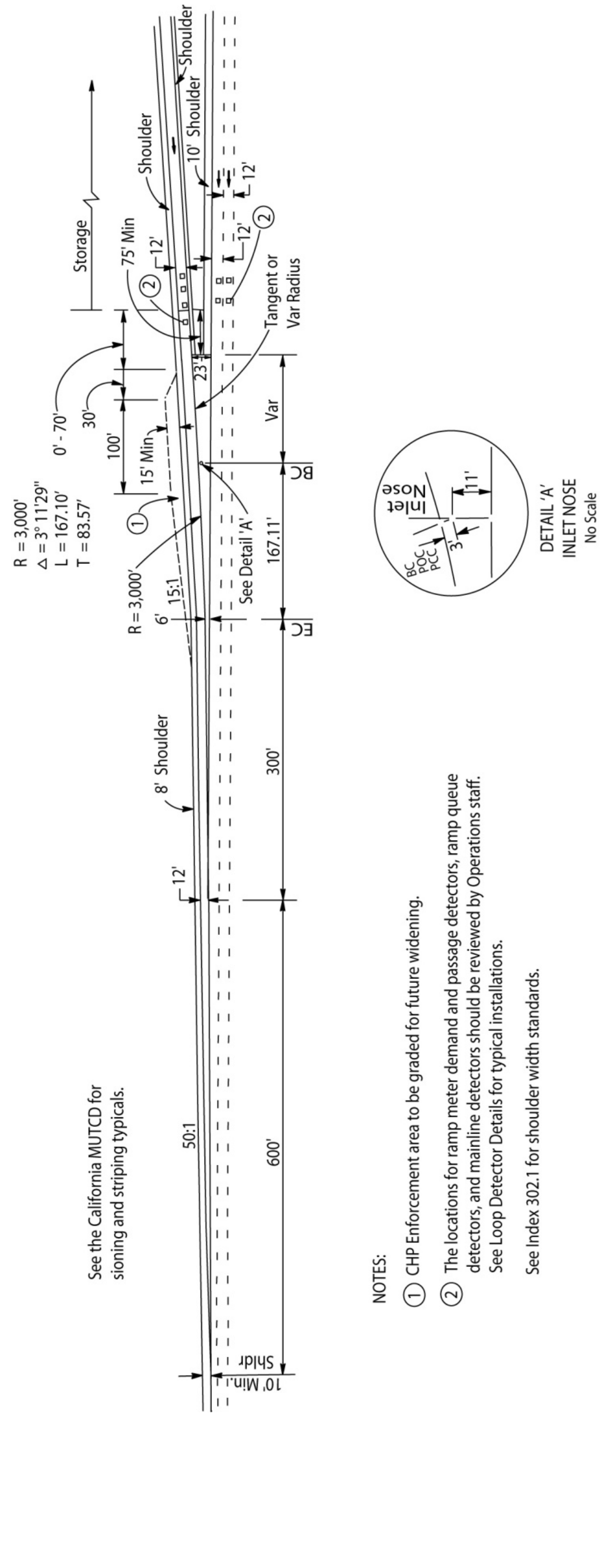
(6) *Two-lane Exit Ramps.* Where design year estimated volumes exceed 1500 equivalent passenger cars per hour, a 2-lane ramp should be provided.

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 Project Delivery Coordinator or District Design Liaison 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

Figure 504.3A
Typical Freeway Entrance
With 1-Lane Ramp Meter



NOTES:

1. CHP Enforcement area to be graded for future widening.
2. The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations staff. See Loop Detector Details for typical installations.

See Index 302.1 for shoulder width standards.

Figure 504.3B
Typical Freeway Entrance Loop Ramp
With 1-Lane Ramp Meter

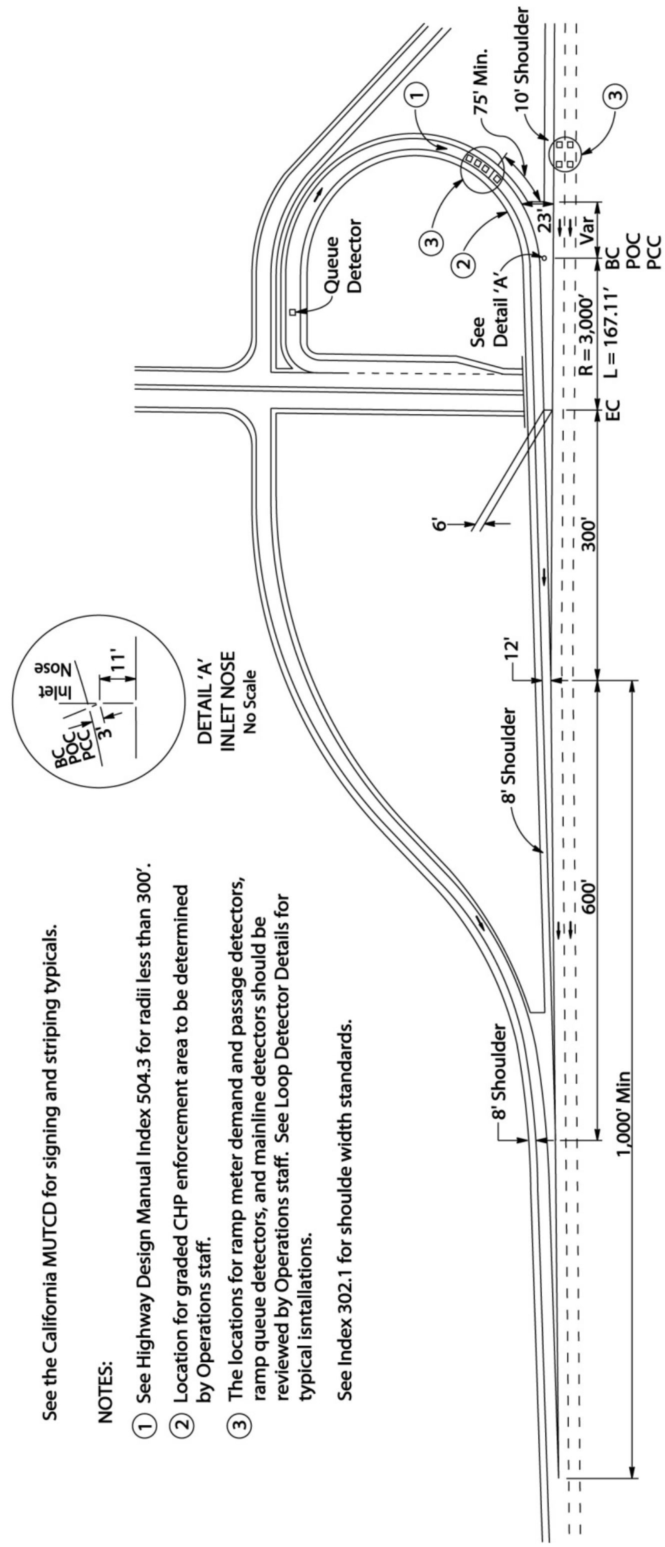
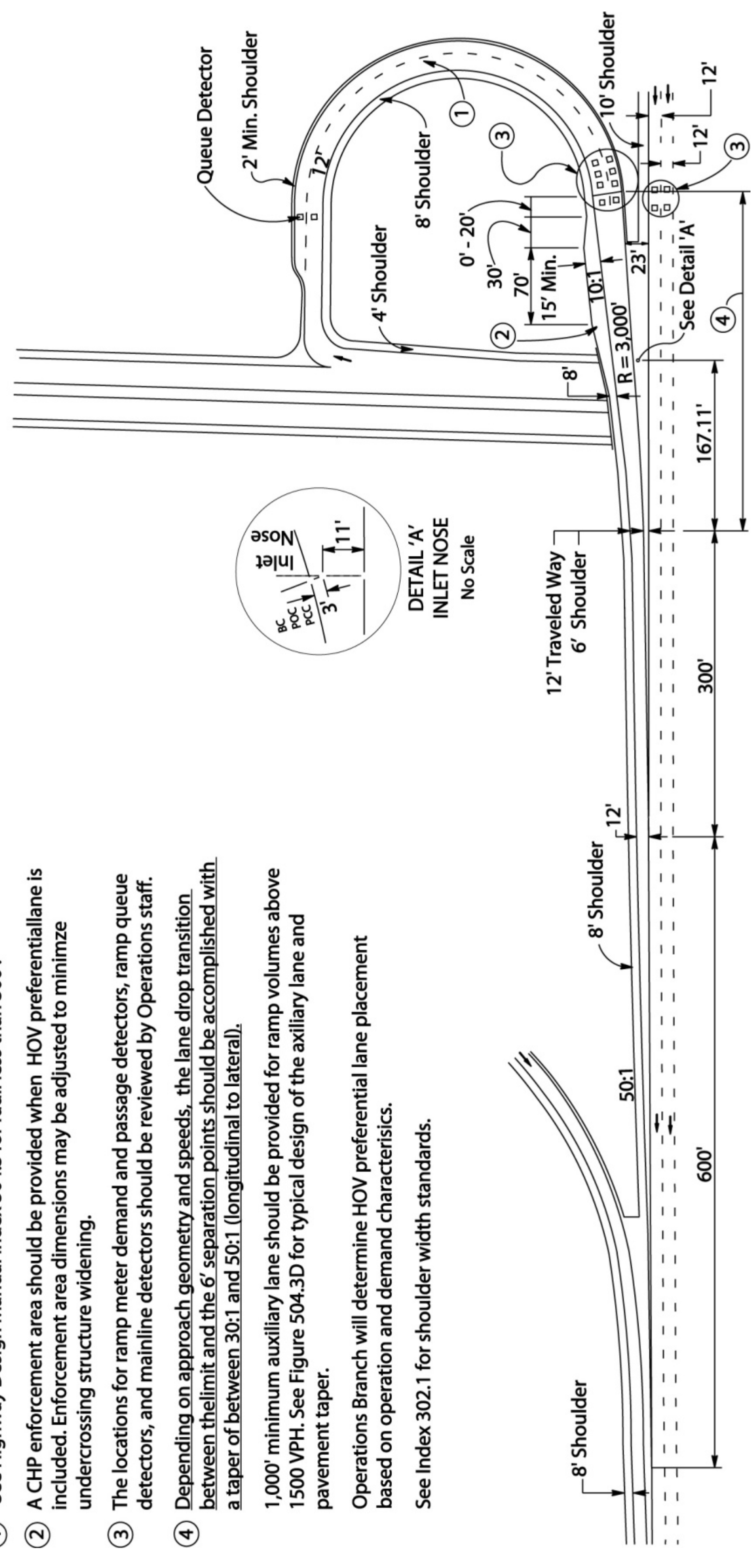


Figure 504.3C
Typical Freeway Entrance Loop Ramp With 2-Lane Ramp Meter



See the California MUTCD for signing and striping typicals.

NOTES:

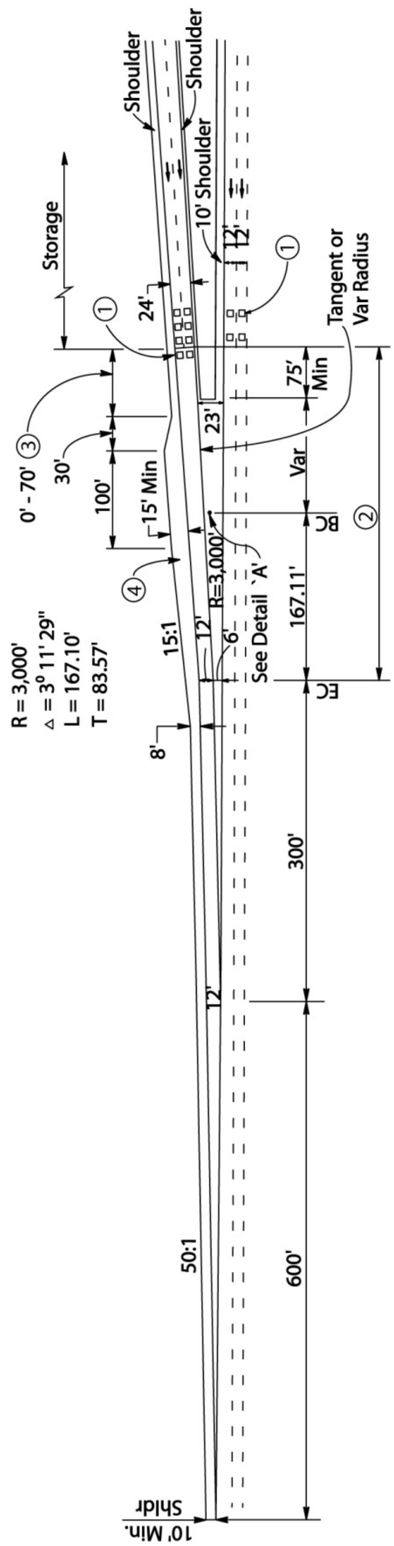
1. See Highway Design Manual Index 504.3 for radii less than 300'.
2. A CHP enforcement area should be provided when HOV preferentiallane is included. Enforcement area dimensions may be adjusted to minimze undercrossing structure widening.
3. The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations staff.
4. Depending on approach geometry and speeds, the lane drop transition between thelimit and the 6' separation points should be accomplished with a taper of between 30:1 and 50:1 (longitudinal to lateral).

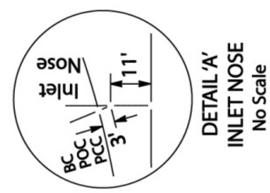
1,000' minimum auxiliary lane should be provided for ramp volumes above 1500 VPH. See Figure 504.3D for typical design of the axiliary lane and pavement taper.

Operations Branch will determine HOV preferential lane placement based on operation and demand characterisics.

See Index 302.1 for shoulder width standards.

Figure 504.3D
Typical Freeway Entrance for Ramp Volumes < 1500 VPH With 2-Lane Ramp Meter





NOTES:

① The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations staff. See LoopDetector Details for typical installations.

② Depending on approach geometry and speed, the lane drop transition between the limit line and the 6' separation point should be accomplished with a taper of between 30:1 and 50:1 (longitudinal to lateral).

③ Use 170' if HOV preferential lane is not metered.

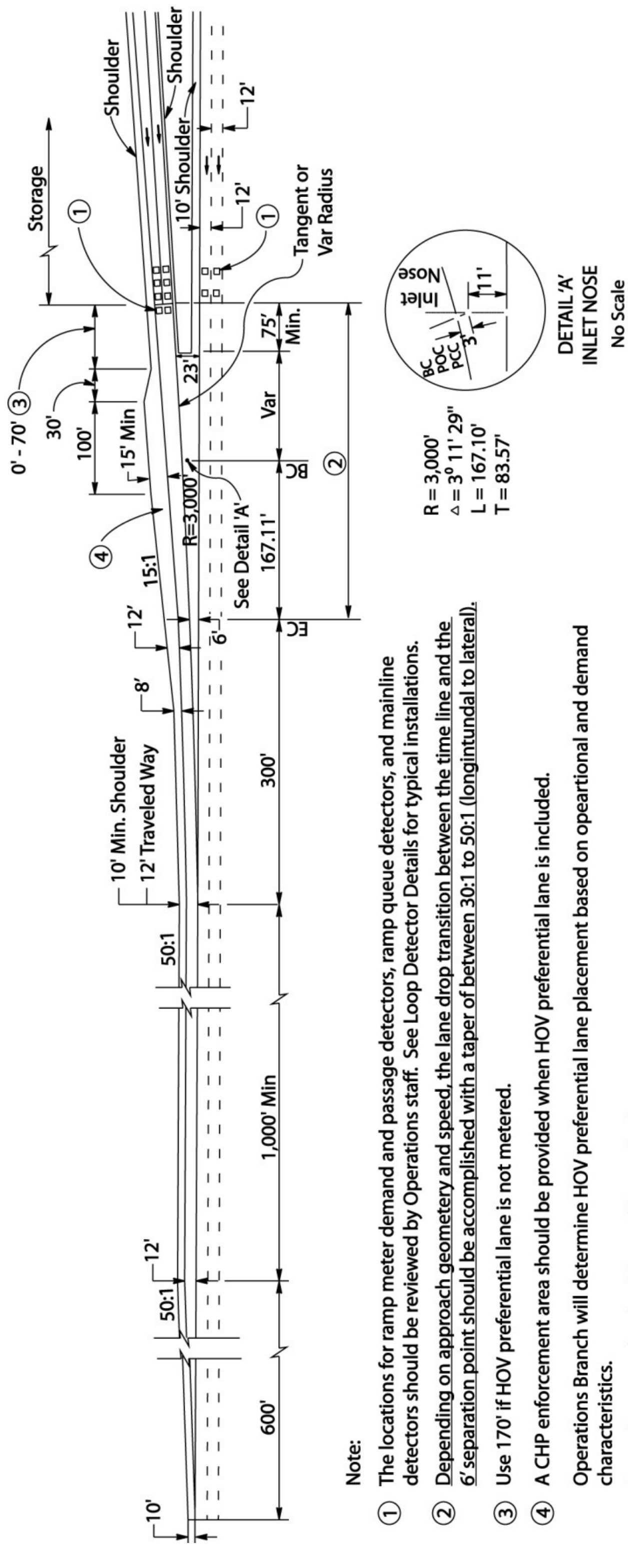
④ A CHP enforcement area should be provided when HOV preferential lane is included.

Use Figure 504.3E when ramp volumes exceed 1500 VPH. Operations Branch will determine HOV preferential placement based on operational and demand characteristics.

See Index 302.1 for shoulder width standards.

See the California MUTCD for signing and striping typicals.

Figure 504.3E
Typical Freeway Entrance for Ramp Volumes > 1500 VPH
With 2-Lane Ramp Meter



Note:

1. The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewed by Operations staff. See Loop Detector Details for typical installations.
2. Depending on approach geometery and speed, the lane drop transition between the time line and the 6' separation point should be accomplished with a taper of between 30:1 to 50:1 (longintundal to lateral).
3. Use 170' if HOV preferential lane is not metered.
4. A CHP enforcement area should be provided when HOV preferential lane is included.

Operations Branch will determine HOV preferential lane placement based on opeartional and demand characteristics.

See Index 302.1 for shoulder width standards.

See the California MUTCD for signing and striping typicals.

Figure 504.3F
Typical Freeway Entrance for Ramp Volumes < 1500 VPH
3-Lane Ramp Meter
(2 mixed-flow lanes + HOV preferential lane)

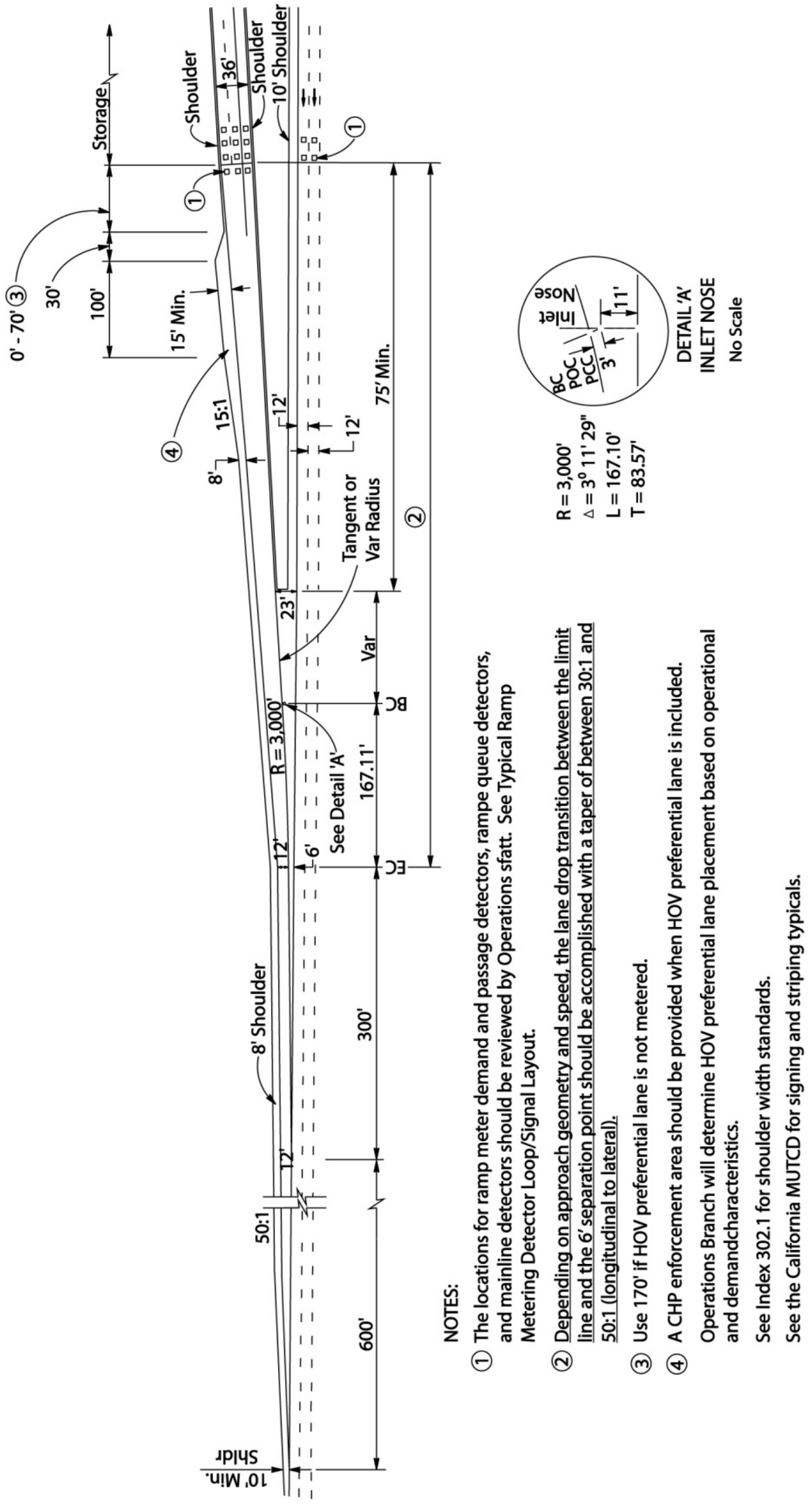
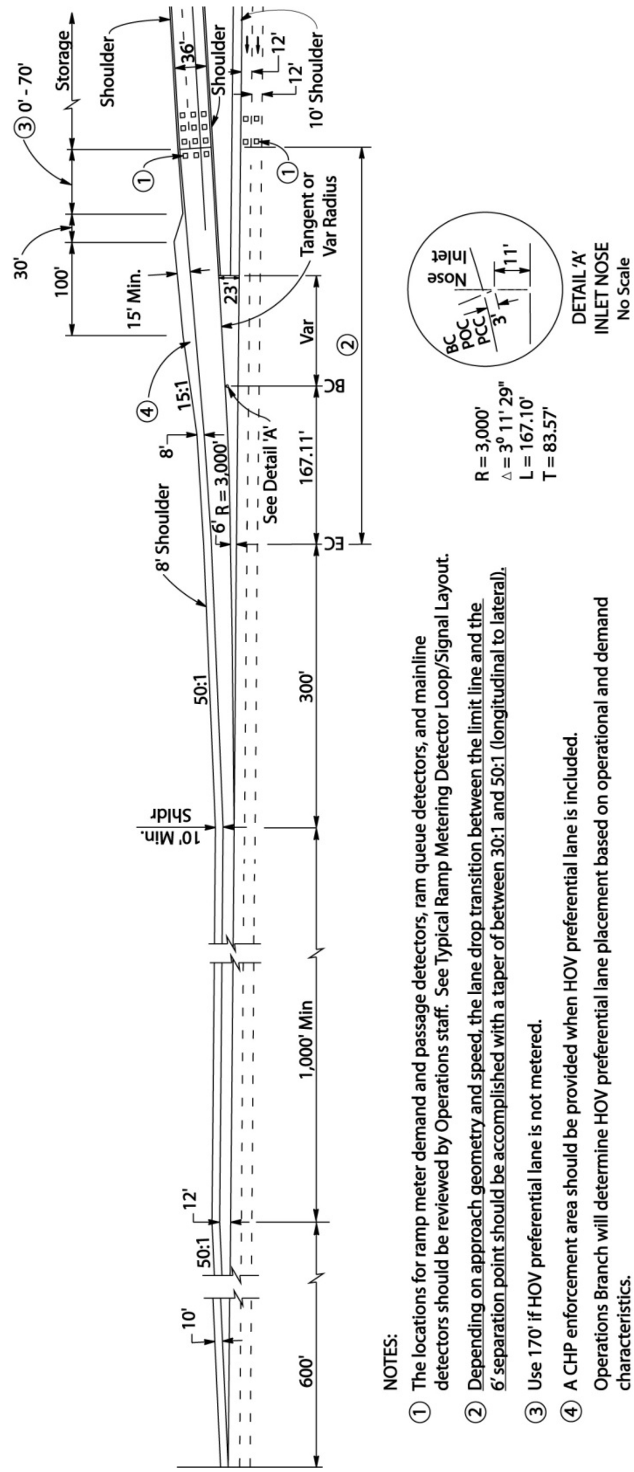


Figure 504.3G
Typical Freeway Entrance for Ramp Volumes > 1500 VPH
3-Lane Ramp Meter
(2 mixed-flow lanes + HOV preferential lane)



NOTES:

① The locations for ramp meter demand and passage detectors, ram queue detectors, and mainline detectors should be reviewed by Operations staff. See Typical Ramp Metering Detector Loop/Signal Layout.

② Depending on approach geometry and speed, the lane drop transition between the limit line and the 6' separation point should be accomplished with a taper of between 30:1 and 50:1 (longitudinal to lateral).

③ Use 170' if HOV preferential lane is not metered.

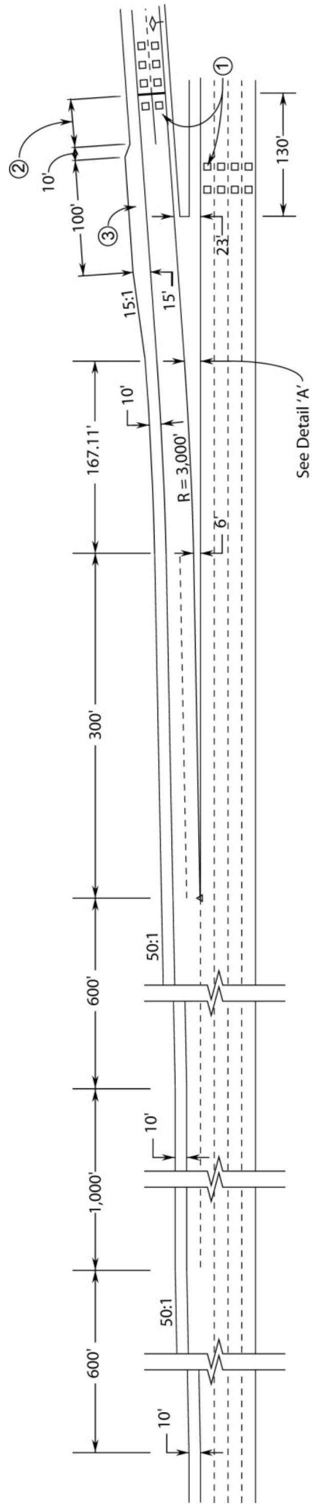
④ A CHP enforcement area should be provided when HOV preferential lane is included.

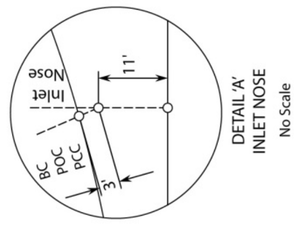
Operations Branch will determine HOV preferential lane placement based on operational and demand characteristics.

See Index 302.1 for shoulder width standards.

See the California MUTCD for signing and striping typicals.

Figure 504.3H
Typical Freeway Connector
2-Lane Meter
(1 mixed-flow lane + HOV preferential lane)





NOTES:

① The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors shold 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 A CHP enforcement area should be provided when HOV preferential lane is included.

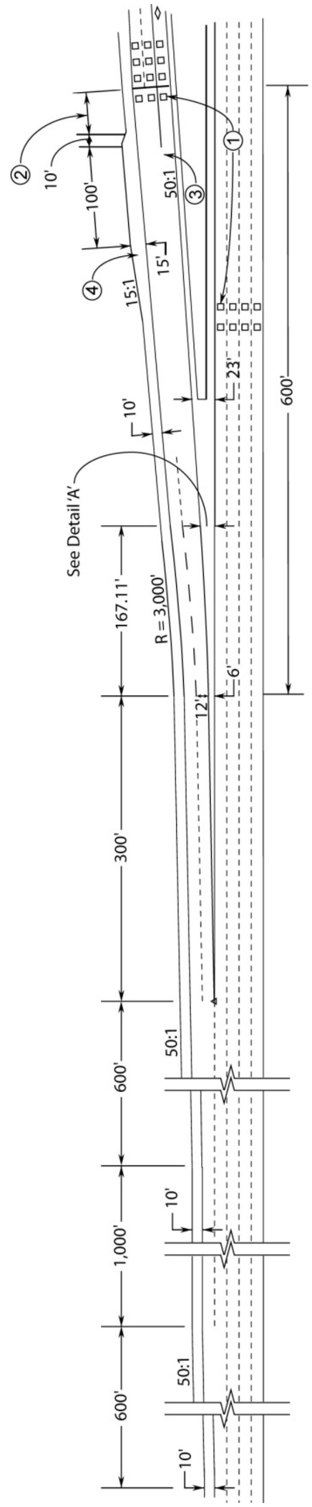
④ Operations staff will determine HOV preferential lane placement based on operational and demand characteristics.

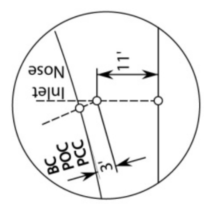
Lane drops should be accomplished over a distance equal to WV, but **the lane drop transition shall be accomplished with at least at a 50:1 (longitudinal to lateral) taper.**

See Index 302.1 for shoulder width standards.

See the California MUTCD for signing and striping typical.

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





DETAIL 'A'
INLET NOSE
No Scale

NOTES:

① The locations for ramp meter demand and passage detectors, ramp queue detectors, and mainline detectors should be reviewd by Operations staff. See Typical Ramp Metering Detector Loop/Signal Layout.

② Use 0' - 70' if HOV prefernial 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 enforceent 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.

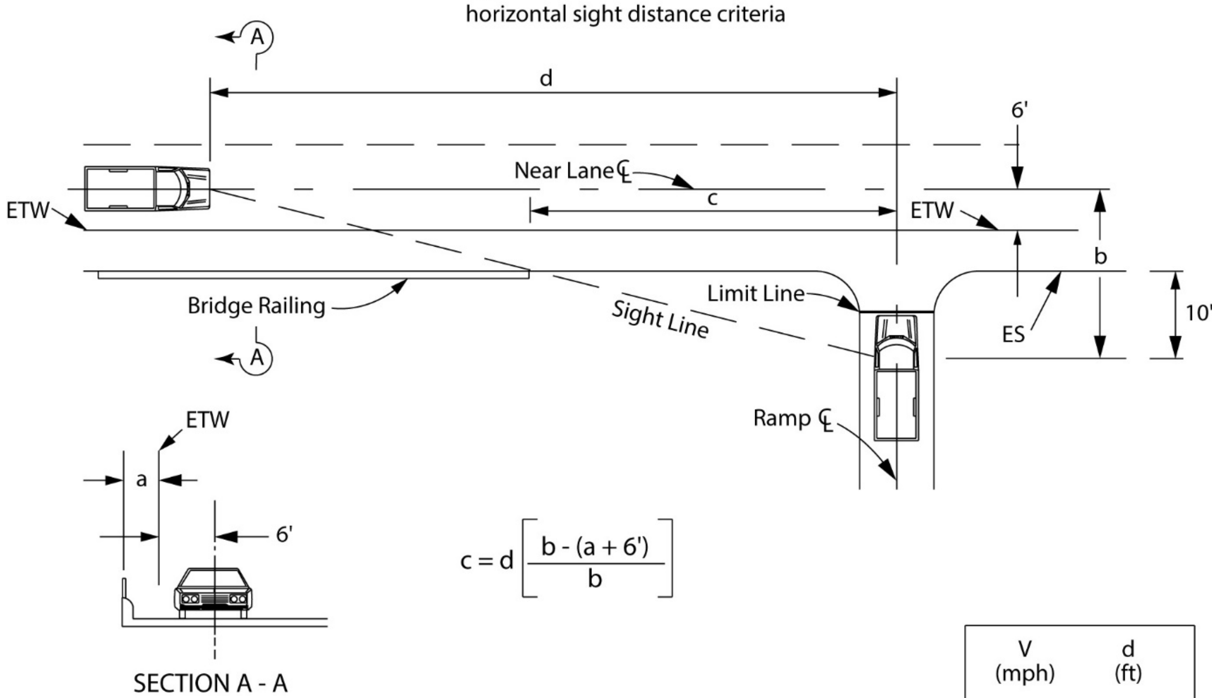
See Index 302.1 for shoulder width standards.

See the California MUTCD for signing and striping typical.

Figure 504.3J

Location of Ramp Intersections on the Crossroads

Unsignalized and based on 7.5 second
horizontal sight distance criteria



$$c = d \left[\frac{b - (a + 6')}{b} \right]$$

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

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 railling.

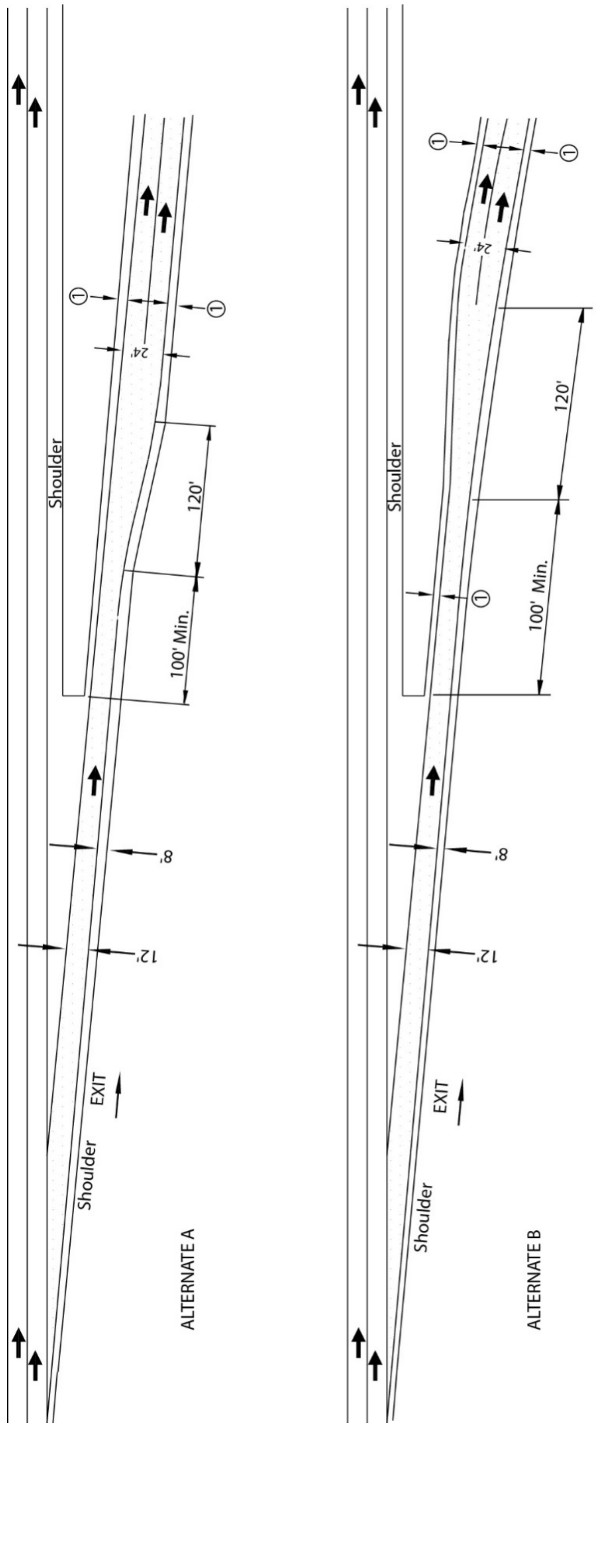
d = Corner Sight distance along highway from intersection. (See Table above) Sight distance is measured from a 3½′ eye height on the ramp to a 4¼′ object height on the crossroad.

V = Anticipated prevailing speed on crossroad.

NOTE:

(1) See the California MUTCD for limit line placement and guidance.

Figure 504.3K
Transition to Two-lane Exit Ramp

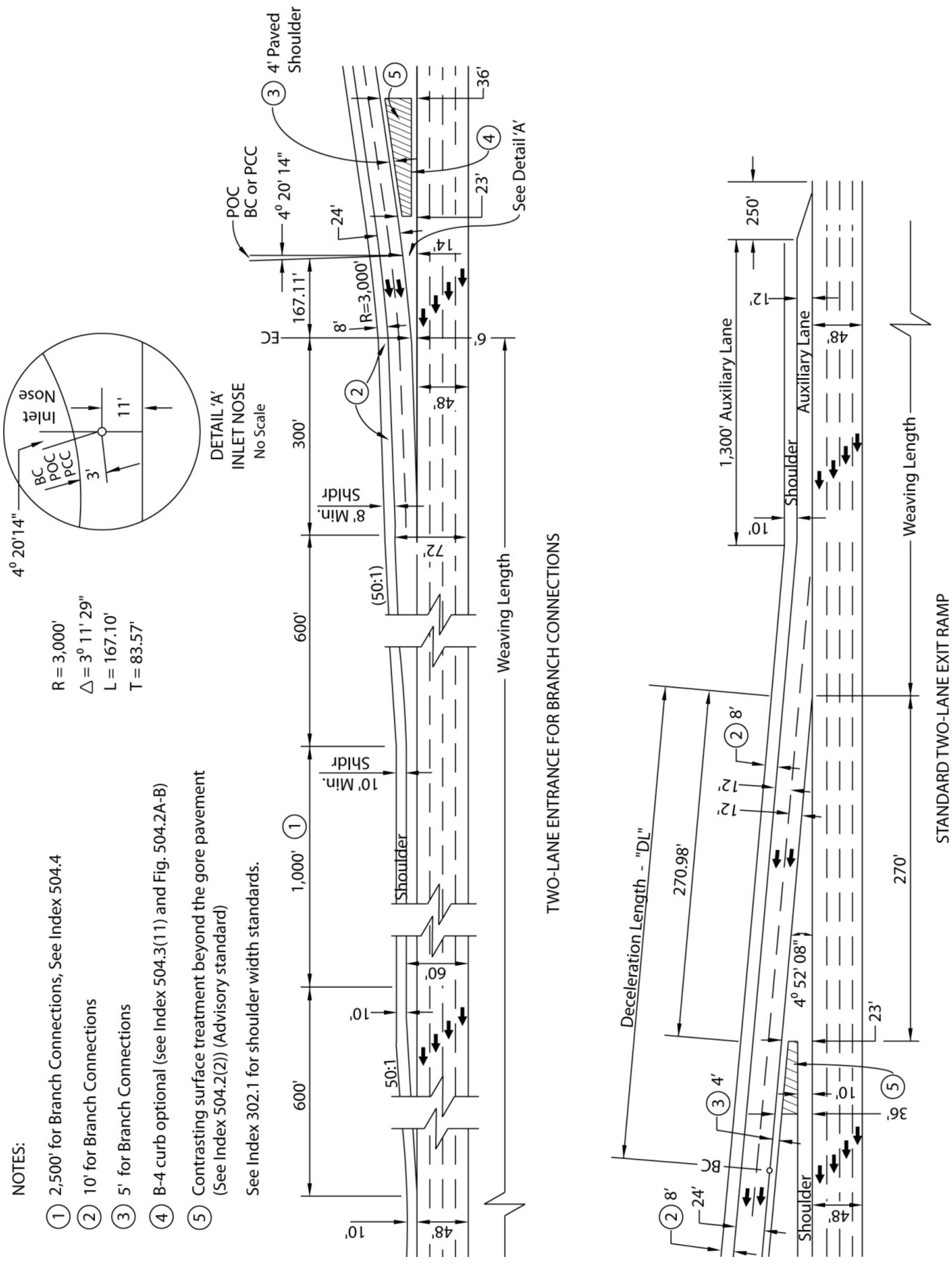


NOTES:

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



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. <u>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.</u> The distance between on-ramp noses will then be controlled by interchange geometry.

(10) Distance Between Successive Exits. <u>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.</u>

(11) Curbs. Curbs should not be used on-ramps except in the following locations:

(a) 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.

(b) 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.

(c) 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.

(d) 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

(1) *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.

(2) *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).

(3) *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.

(4) *Shoulder Width.*

(a) 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)).

(b) Three-lane Connections--**The width of shoulders on three-lane connectors shall be 10 feet on both the left and right sides.**

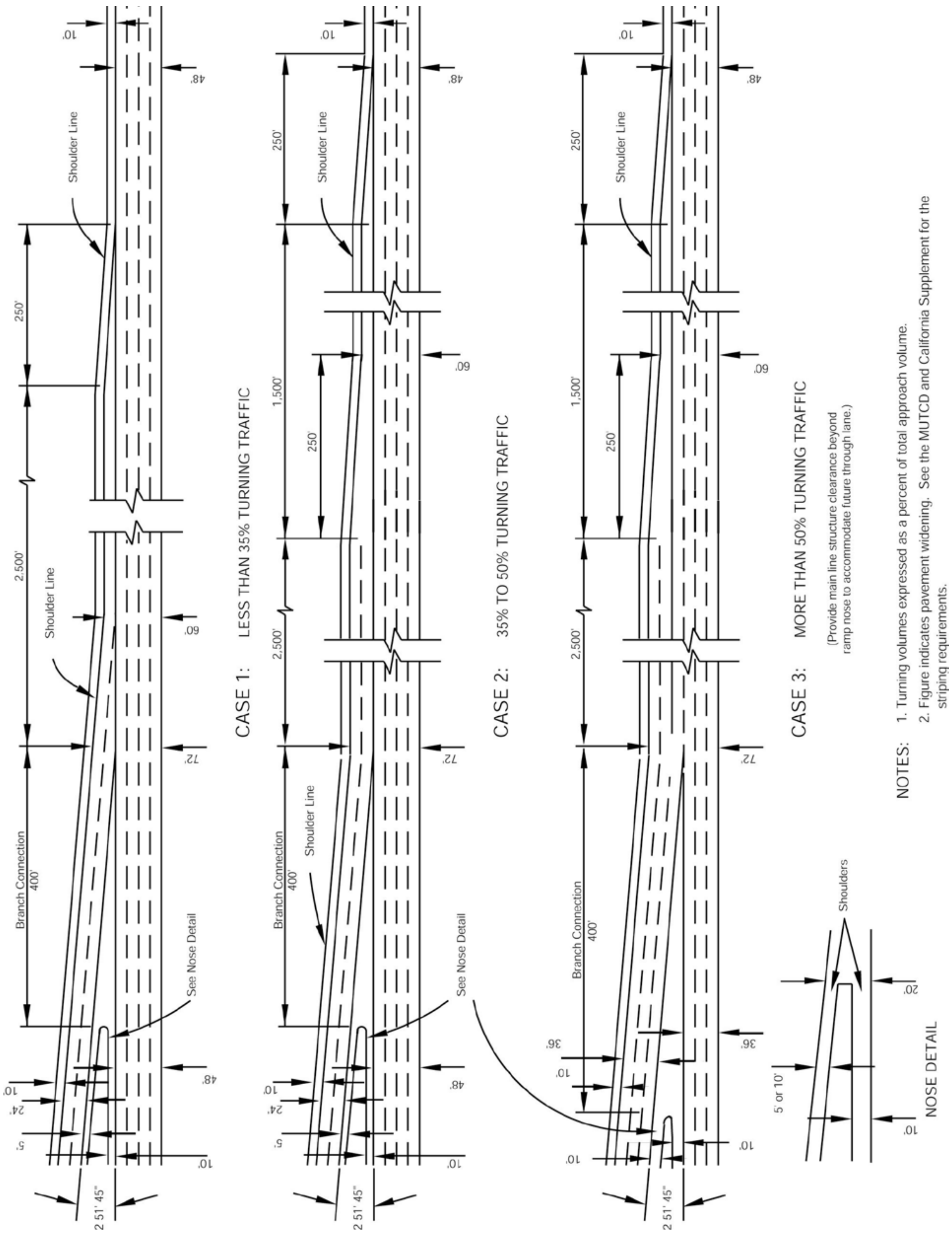
(5) *Single-lane Connections.* Freeway-to-freeway connectors may be single lane or multilane. Where design year volume is between 900 and 1500 equivalent passenger cars per hour, initial construction should provide a single lane connection with the capability of adding an additional lane. Single lane 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

Figure 504.4
Diverging Branch Connections



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 Project Delivery 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 more than 35 percent of the freeway traffic is turning, consideration may be given to reducing the number of lanes. No decision to reduce the number of lanes should be made without the approval of the District Traffic Operations Unit. Additionally, adequate structure clearance (both horizontal and vertical) should be provided to accommodate future construction of the dropped lane if required.

Where the reduction in traffic volumes is sufficient to warrant a decrease in the basic number of lanes, a preferred location for the lane drop is beyond the influence of an interchange and preferably at least one-half mile from the nearest exit or inlet nose. It is desirable to drop the right lane on tangent alignment with a straight or sag profile so vehicles can merge left with good visibility to the pavement markings in the merge area (see Index 201.7).

504.7 Weaving Sections

A weaving section is a length of one-way roadway where vehicles are crossing paths, changing lanes, or merging with through traffic as they enter or exit a freeway or collector-distributor road.

A single weaving section has an inlet at the upstream end and an exit at the downstream end. A multiple weaving section is characterized by more than one point of entry followed by one or more points of exit.

A rough approximation for adequate length of a weaving section is one foot of length per weaving vehicle per hour. This rate will approximately provide a Level of Service (LOS) C.

There are various methods for analyzing weaving sections. Two methods which provide valid results are described below.

The Leisch method, which is usually considered the easiest to use, is illustrated in Figure 504.7A. This method was developed by Jack Leisch &

Associates and may be used to determine the length of weaving sections for both freeways and collector-distributor roads. The Leisch weaving charts determine the level of service for the weaving volumes for the length of the weaving section from the first panel on the lower left of the chart. The analysis is dependent on whether the section is balanced or unbalanced, as defined in Figure 504.7B. The level of service for the total

volume over all lanes of the weaving section is then found from the panels on the right of the chart. The weaving chart should not be extrapolated.

Pages 234-238 of the 1965 Highway Capacity Manual (HCM) provide a method for determining the adequacy of weaving sections near single lane ramps. It is often referred to as the LOS D method. This method is also documented in Traffic Bulletin 4 which is available from the District Division of Traffic Operations. The LOS D method can be used to project volumes along a weaving section. These volumes can be compared to the capacities along the same weaving section.

Volumes in passenger car equivalents per hour (PCEPH) should be adjusted for freeway grade and truck volumes. Table 504.7C and Figures 504.7D and E are reprinted from the 1965 HCM and provide information regarding vehicle distribution by lane.

The results obtained from Figure 504.7A (the Leisch Method) for single-lane ramps with an auxiliary lane and weaving rates exceeding 2500 PCEPH should be checked using the LOS D method.

Weaving capacity analyses other than those described above should not be used on California highways. Other methods, such as the one contained in the 1994 HCM, may not always produce accurate results.

The criteria 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.

Weaving sections in urban areas should be designed for LOS C or D. Weaving sections in rural areas should be designed for LOS B or C. Design rates for lane balanced weaving sections where at least one ramp or connector will be two lanes should not result in a LOS lower than the middle of LOS D using Figure 504.7A. Mainline through capacity is optimized when weaving 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. <u>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.</u> **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

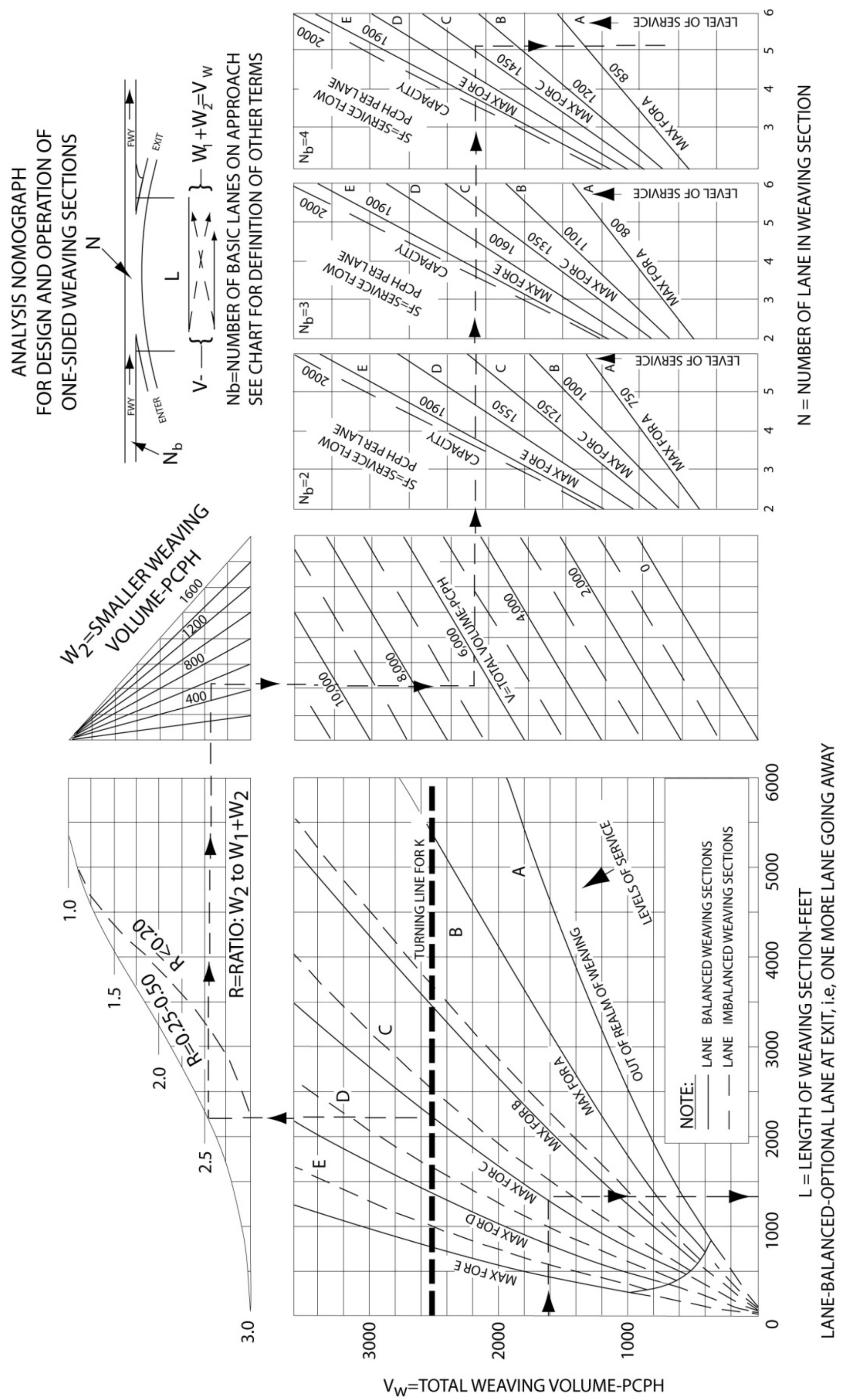
preclude driveways or local roads within the ramp intersection. This access control would limit the volume of traffic and the number of phases at the intersection of the ramp and local facility, thereby optimizing capacity and operation of the ramp. Through a combination of access control and the use 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.

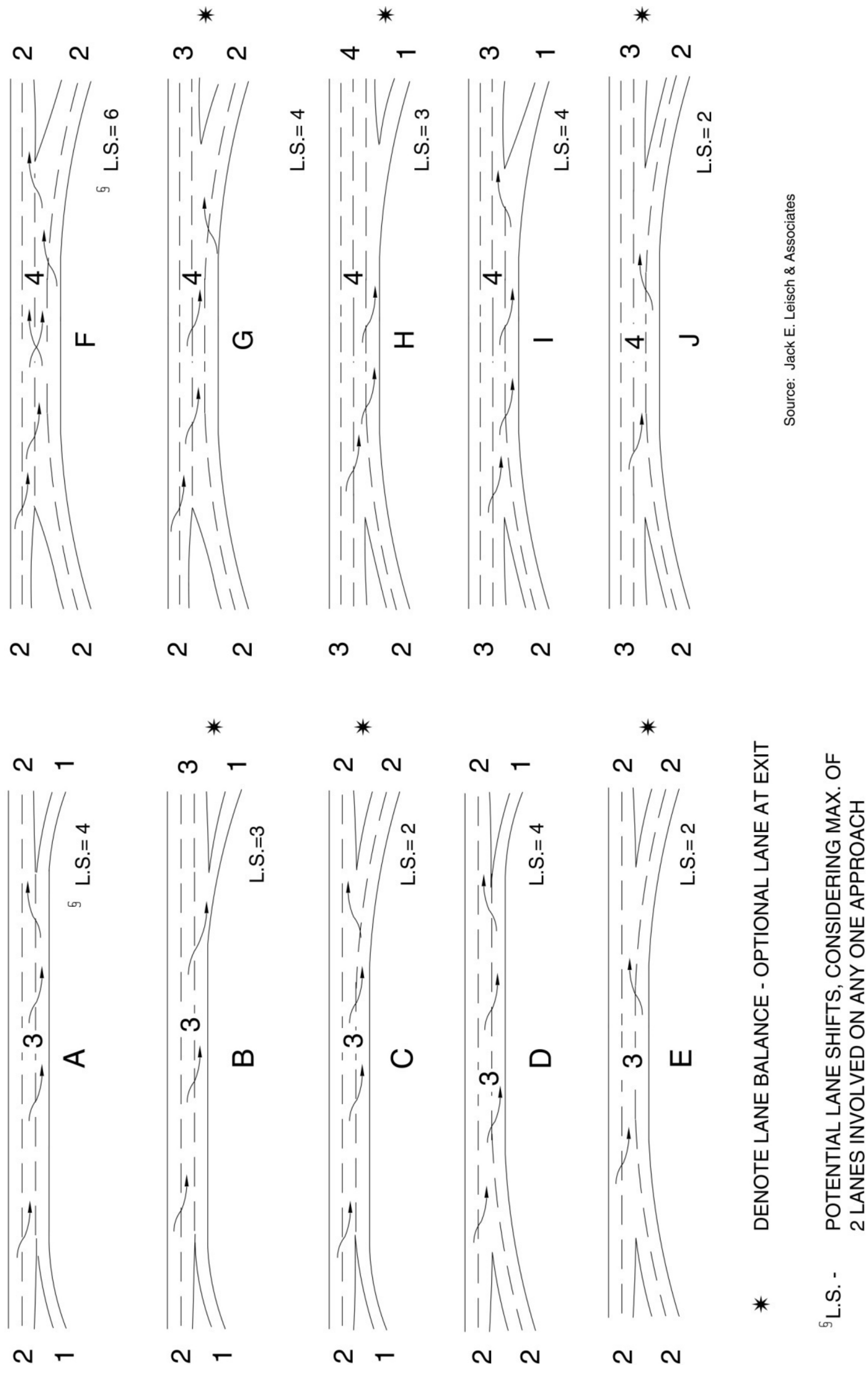
Figure 504.7A
Design Curve for Freeway and Collector Weaving



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 Vw) 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 Vw 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.

Figure 504.7B
Lane Configuration of Weaving Sections



Source: Jack E. Leisch & Associates

Table 504.7C

Percent of Through Traffic Remaining in Outer Through Lane (Level of Service D Procedure)

Total Volume of Through Traffic, One Direction (vph)	Approximate Percentage of Through[1] Traffic Remaining in the Outer Through Lane in the Vicinity of Ramp Terminals at Level of Service D		
	8-Lane[2] Freeway	6-Lane[3] Freeway	4-Lane[4] Freeway
6500 and over	10	-	-
6000 - 6499	10	-	-
5500 - 5999	10	-	-
5000 - 5499	9	-	-
4500 - 4999	9	18	-
4000 - 4499	8	14	-
3500 - 3999	8	10	-
3000 - 3499	8	6	40
2500 - 2999	8	6	35
2000 - 2499	8	6	30
1500 - 1999	8	6	25
Up to 1499	8	6	20

NOTES:

(1) Traffic not involved in a ramp movement within 4,000 feet in either direction.

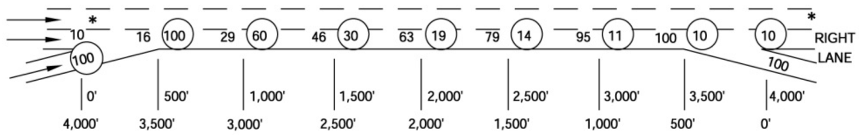
(2) 4 lanes one-way.

(3) 3 lanes one-way.

(4) 2 lanes one-way.

Figure 504.7D
Percentage Distribution of On- and Off-ramp Traffic in Outer Through Lane and Auxiliary Lane
(Level of Service D Procedure)

CASE I - SINGLE - LANE ON- AND OFF-RAMPS WITHOUT AUXILIARY LANE
(THIS CHART MAY BE USED REGARDLESS OF ACTUAL SPACING BETWEEN ON- AND OFF-RAMPS, BUT AS NOTED BELOW* CAUTION MUST BE EXERCISED IN USING THESE VALUES.)



CASE II - SINGLE - LANE - ON- AND OFF-RAMPS WITH AUXILIARY LANE**

(A) L = 1,000'

EXAMPLE:

GIVEN: L = 1,000'

PORTION OF V_1 THROUGH

(FROM TABLE 504.7C = 475 VPH

ON-RAMP = 1,000 VPH

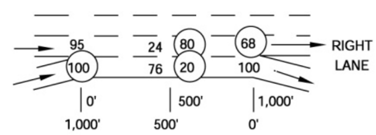
OFF-RAMP = 1,200 VPH

ON-RAMP TO OFF-RAMP = 0

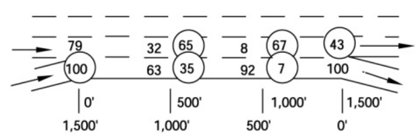
FIND: V_1 (VOL. IN OUTER THROUGH LANE) @ 500' =

475 + (0.80)(1,000) + (0.24)(1,200) =

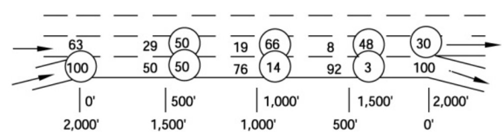
1,563 VPH



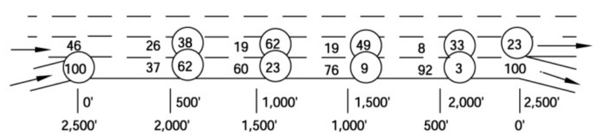
(B) L = 1,500'



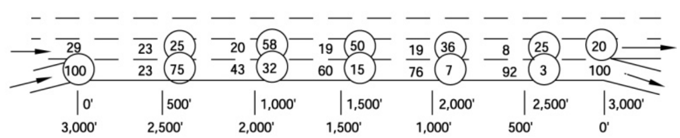
(C) L = 2,000'



(D) L = 2,500'



(E) L = 3,000'



CIRCLED VALUES 0 INDICATE PERCENTAGE OF ON-RAMP TRAFFIC IN LANE SHOWN. UNCIRCLED VALUES INDICATE PERCENTAGE OF OFF-RAMP TRAFFIC IN LANE SHOWN. (REMAINING PORTION OF TRAFFIC IS IN LANE(S) TO LEFT OF OUTER THROUGH LANE.)

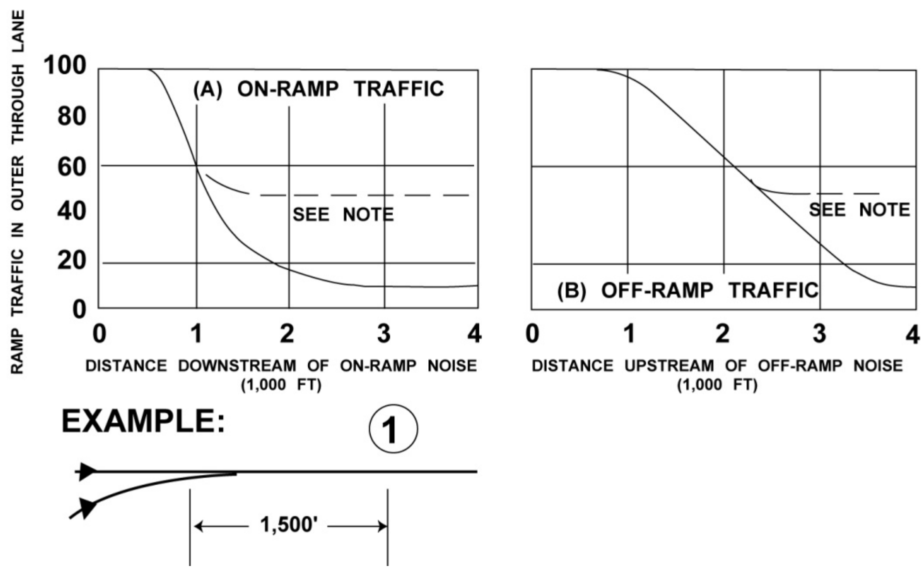
THESE PERCENTAGES ARE NOT NECESSARILY THE DISTRIBUTIONS UNDER FREE FLOW OR LIGHT RAMP TRAFFIC, BUT UNDER PRESSURE OF HIGH VOLUMES IN THE RIGHT LANES AT THE POINT BEING CONSIDERED AND WITH ROOM AVAILABLE IN OTHER LANES.

* MINIMUM % IN RIGHT LANE CANNOT BE LESS THAN % OF THROUGH TRAFFIC IN RIGHT LANE AS DETERMINED FROM TABLE 504.7C (SEE NOTE, FIG. 5047E).

** SEE FIGURES 504.2A AND 504.2B FOR METHOD OF MEASURING LENGTH L (WEAVING LENGTH).

Figure 504.7E

Percentage of Ramp Traffic in the Outer Through Lane (No Auxiliary Lane) (Level of Service D Procedure)



A - NORMAL CALCULATION

2 LANES ONE-WAY

"THROUGH TRAFFIC" = 2,400 VPH

"ON-RAMP" = 800 VPH

AMOUNT IN THE OUTER THROUGH LANE AT 1

THROUGH (FROM TABLE 504.7C) = 0.30 X 2,400 = 720

ON-RAMP (FROM CHART ABOVE) = 0.30 X 800 = 240

960

B - CHECK CALCULATIONS

BECAUSE % IN THE OUTER THROUGH LANE AT 1,500' IS BELOW DASHED LINE, RECALCULATE ASSUMING ON-RAMP TRAFFIC IS THROUGH TRAFFIC.

AMOUNT IN THE OUTER THROUGH LANE AT 1

THROUGH (FROM TABLE 504.7C) 0.40 X 3,200 = 1,280

SINCE CALCULATION B (1,280) IS GREATER THAN

CALCULATION A (960) USE 1,280.

*THESE PERCENTAGES ARE NOT NECESSARILY THE DISTRIBUTIONS UNDER FREE FLOW OR LIGHT RAMP TRAFFIC, BUT UNDER PRESSURE OF HIGH VOLUMES IN THE RIGHT LANES AT THE LOCATION BEING CONSIDERED AND WITH AVAILABLE ROOM IN OTHER LANES.

NOTE:

IF RAMP PERCENTAGE IN THE OUTER THROUGH LANE AT POINT UNDER CONSIDERATION IS BELOW DASHED LINE, THEN AMOUNT IN THE OUTER THROUGH LANE SHOULD BE RECALCULATED ASSUMING RAMP TRAFFIC IS THROUGH TRAFFIC. USE HIGHER VALUE. SEE EXAMPLE ABOVE.

Figure 504.8

Typical Examples of Access Control at Interchanges

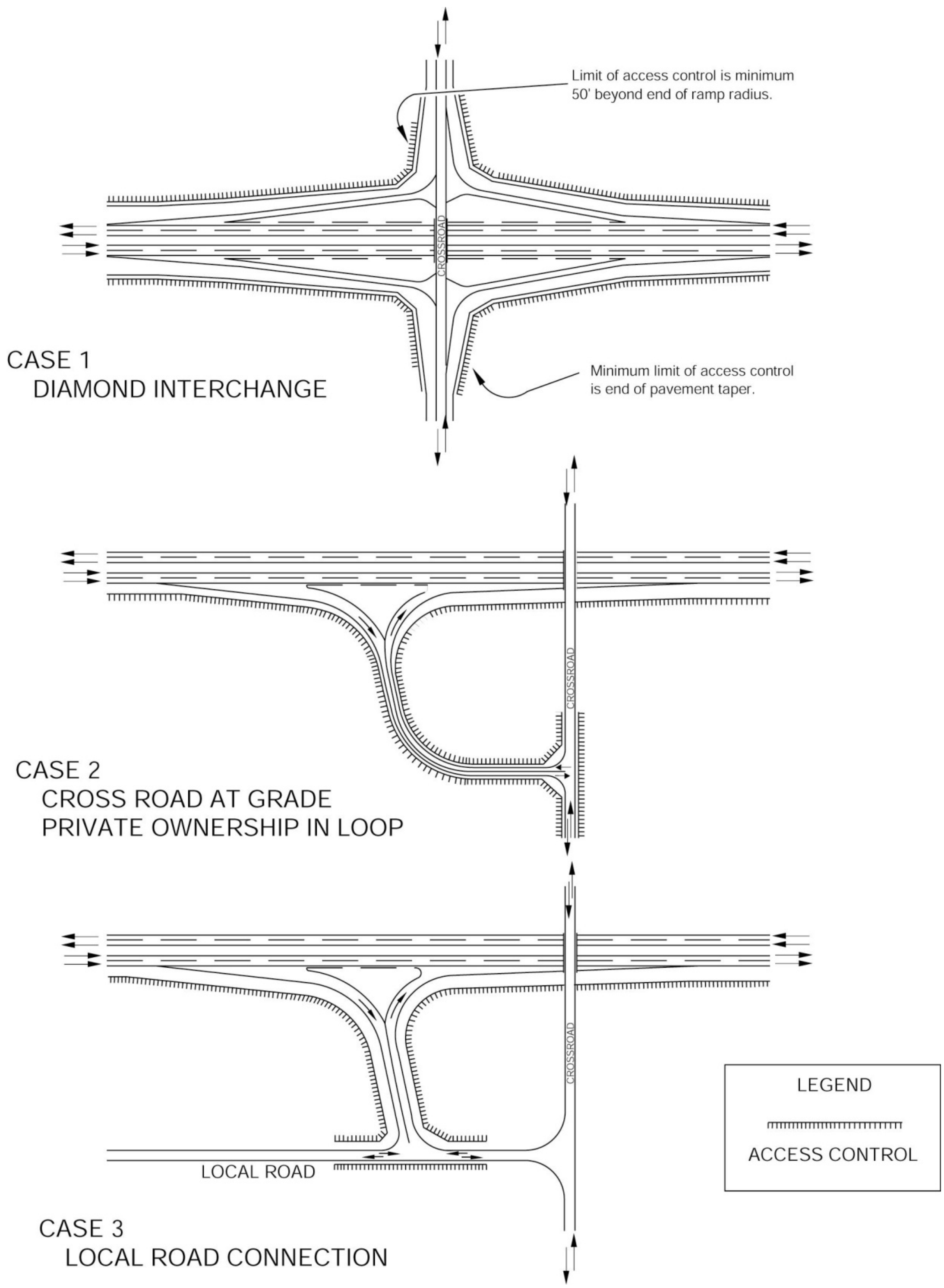
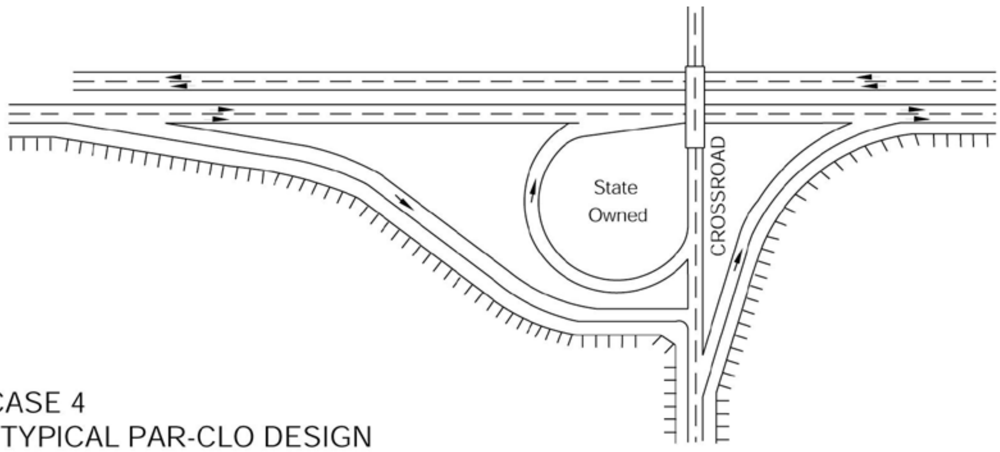
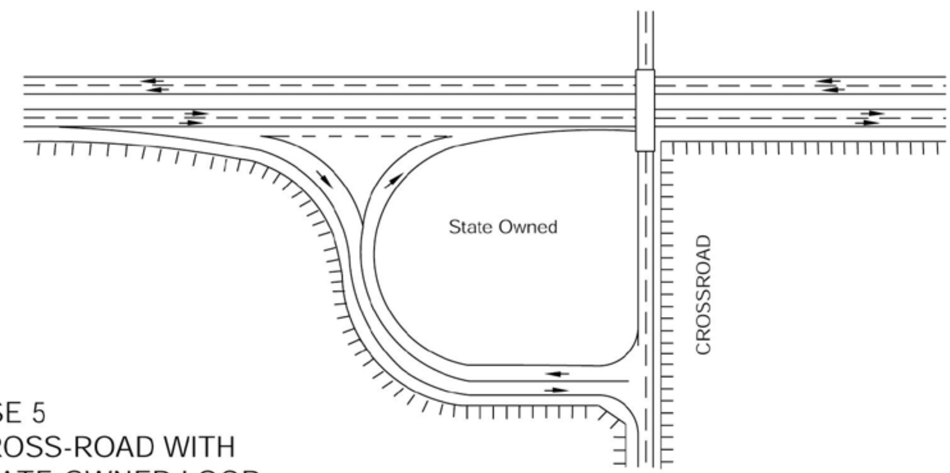


Figure 504.8 (cont.)

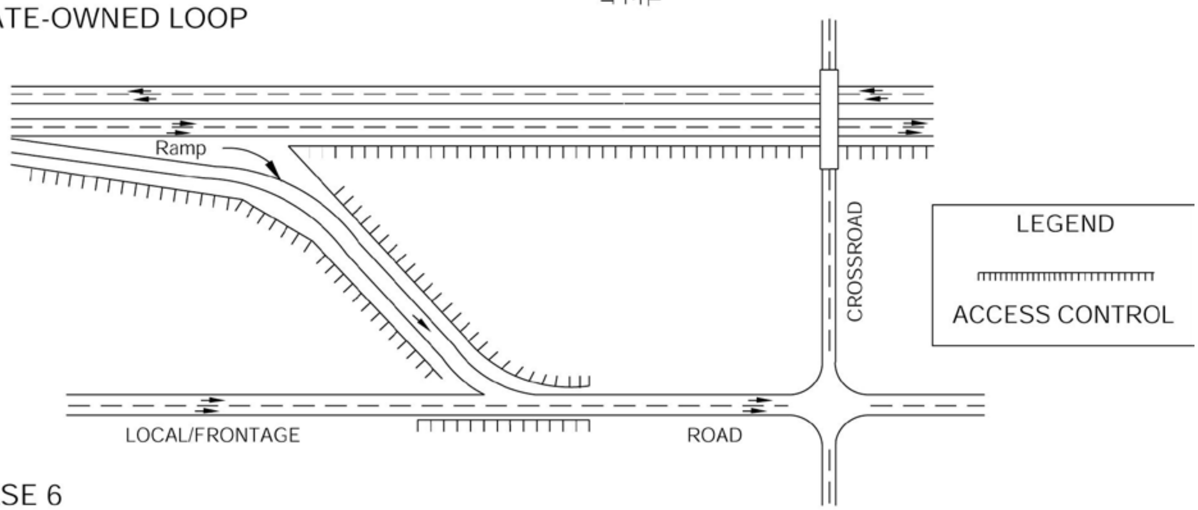
Typical Examples of Access Control at Interchanges



CASE 4
TYPICAL PAR-CLO DESIGN



CASE 5
CROSS-ROAD WITH
STATE-OWNED LOOP



CASE 6
ONE-WAY FRONTAGE ROAD

CHAPTERS 600 – 670 PAVEMENT ENGINEERING

CHAPTER 600 GENERAL ASPECTS

Topic 601 - Introduction

Pavement engineering involves the determination of the type and thickness of pavement surface course, base, and subbase layers that in combination are cost effective and structurally adequate for the projected traffic loading and specific project conditions. This combination of roadbed materials placed in layers above the subgrade (also known as basement soil) is referred to as the "pavement" or the "pavement structure".

The Department guidelines and standards for pavements described in this manual are based on extensive engineering research and field experience, including the following:

- Theoretical concepts in pavement engineering and analysis.
- Data obtained from test track studies and experimental sections.
- Research on materials characteristics, testing methods, and equipment.
- Observation of performance throughout the State and the nation.

The pavement should be engineered using the standards and guidance described in this manual to ensure consistency throughout the State and provide a pavement structure that will have adequate strength, ride quality, and durability to carry the projected traffic loads for the design life of each project. The final pavement structure for each project should be based on a thorough investigation of specific project conditions including subgrade soils and structural materials, environmental conditions, projected traffic, cost effectiveness, and the performance of other pavements in the same area or similar climatic and traffic conditions. These factors are discussed in Chapter 610 of this manual.

The guidelines and standards found in this manual should be considered minimum standards and should not preclude sound engineering judgment based on experience and knowledge of the local conditions. Sound engineering judgment must still be used to determine if more stringent standards are required.

Topic 602 – Pavement Structure Layers

Index 602.1 Description

Pavement structures are comprised of one or more layers of select materials placed above the subgrade. The basic pavement layers of the roadway are shown in Figure 602.1 and discussed below.

(1) Subgrade. Also referred to as basement soil, the subgrade is that portion of the roadbed consisting of native or treated soil on which surface course, base, subbase, or a layer of any other material is placed. Subgrade may be composed of either in-place material that is exposed from excavation, or embankment material that is placed to elevate the roadway above the surrounding ground. Subgrade soil characteristics are discussed in Topic 614.

(2) Subbase. Unbound or treated aggregate/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. The subbase generally consists of lower quality materials than the base but better than the subgrade soils. Subbase may not be needed in areas with higher quality subgrade (California R-value > 40) or where it is more cost effective to build a thicker base layer. Further discussion on subbase materials and concepts can be found in Chapter 660.

(3) Base. Select, 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. Base may be one or multiple layers treated with cement, asphalt or other binder material, or may consist of untreated aggregate. In some cases, the base may include a drainage layer to drain water that seeps into the base. The aggregate in base is typically a higher quality material than that used in subbase. Further discussion on base materials and concepts can be found in Chapter 660.

(4) *Surface Course.* One or more layers of the pavement structure engineered to accommodate and distribute traffic loads, provide skid resistance, minimize disintegrating effects of climate, reduce tire/pavement noise, improve surface drainage, and minimize infiltration of surface water into the underlying base, subbase and subgrade. Sometimes referred to as the surface layer, the surface course may be composed of a single layer, constructed in one or more lifts of the same material, or multiple layers of different materials.

Depending on the type of base or subbase layers, surface courses are used to characterize pavements into the following three categories:

(a) Flexible Pavements. These are pavements engineered to bend or flex when loaded. Flexible pavements transmit and distribute traffic loads to the underlying layers. The highest quality layer is the surface course, which typically consists of one or more layers of asphalt binder mixes and may or may not incorporate underlying layers of base and/or subbase. These types of pavements are called "flexible" because the total pavement structure bends (or flexes) to accommodate deflection bending under traffic loads. Procedures for flexible pavements can be found in Chapter 630.

(b) Rigid Pavements. These are pavements with a rigid surface course typically a slab of Portland cement concrete (or a variety of specialty hydraulic cement concrete mixes used for rapid strength concrete) over underlying layers of stabilized or unstabilized base or subbase materials. These types of pavements rely on the substantially higher stiffness of the concrete slab to distribute the traffic loads over a relatively wide area of underlying layers and the subgrade. Some rigid concrete slabs have reinforcing steel to help resist cracking due to temperature changes and repeated loading. Procedures for rigid pavements can be found in Chapter 620.

(c) Composite Pavements. These are pavements comprised of both flexible (asphalt binder mixes) and rigid (cement concrete) layers over underlying layers of stabilized or unstabilized base or subbase materials. Currently, for purposes of the procedures in this manual, only pavements with a flexible layer over a rigid surface layer are considered to be composite pavements. In California, such pavements consist mostly of existing rigid pavements (typically Portland cement concrete) that have had a flexible surface course overlay such as hot mix asphalt (HMA) (formerly known as asphalt concrete), open graded friction course (OGFC) (formerly known as open graded asphalt concrete), or rubberized hot mix asphalt (RHMA) (formerly known as rubberized asphalt concrete). See Chapter 640 for additional information on composite pavements.

(5) *Non-Structural Wearing Course.* On some pavements, a non-structural wearing course is placed to protect the surface course from wear and tear from tire/pavement interaction, the weather, and other environmental factors. Examples of non-structural wearing courses include OGFC, various types of surface seals, and added surface course thickness to allow for chain wear or grinding. Although non-structural wearing courses are not given a structural value in the procedures and tables found in this manual, they will improve the service life of the pavement by protecting it from traffic and environmental effects.

(6) *Others.* Depending on the type of pavement built and the subgrade or existing soil conditions encountered, additional layers may

be included in the pavement. Some of these layers include:

(a) Interlayers can be used between pavement layers or within pavement layers to reinforce pavement and/or improve resistance to reflective cracking of the pavement structure.

(b) Bond Breakers are used to prevent bonding between two pavement layers such as rigid pavement surface course to a stabilized base.

(c) Tack Coats are used to bond a layer of asphalt binder mix to underlying existing pavement layers or between layers of asphalt binder mixes where multiple lifts are required.

(d) Prime Coats can be used on aggregate base prior to paving for better bonding and to act as water proofing of the aggregate base.

(e) Leveling Courses are used to fill and level surface irregularities and ruts before placing overlays.

Topic 603 – Types of Pavement Projects

603.1 New Construction

New construction is the building of a new facility. This includes new roadways, interchanges or grade separation crossings, and new parking lots or safety roadside rest areas.

603.2 Widening

Widening projects involve the construction of additional width to improve traffic flow and increase capacity on an existing highway facility. Widening may involve adding lanes (including transit or bicycle lanes), shoulders, pullouts for maintenance/transit traffic; or widening existing lane, shoulder or pullouts.

It is often not cost-effective or desirable to widen a highway without correcting for bad ride and major structural problems in adjacent pavements when that work is needed. Therefore, on widening projects such as lane/shoulders additions, auxiliary lanes, climbing or passing lanes, etc., the existing adjacent pavement condition should be investigated to determine if rehabilitation or pavement preservation is warranted. If warranted, combining rehabilitation or pavement preservation work with widening is strongly encouraged. Combining widening with work on existing pavement can minimize traffic delay and long-term costs. For example, grinding the adjoining rigid pavement lane next to the proposed widening can improve constructability and provide a smoother pavement surface for the widening. For flexible pavement projects, a minimum of 0.15 foot overlay over the widening and existing pavement should be used to eliminate pavement joints which are susceptible to water intrusion and early fatigue failure.

Additional guidance and requirements on widening existing facilities, including possible options as well as certain circumstances that may justify adding rehabilitation or pavement preservation work to widening, or deferring it, are discussed in Index 612.3.

603.3 Pavement Preservation

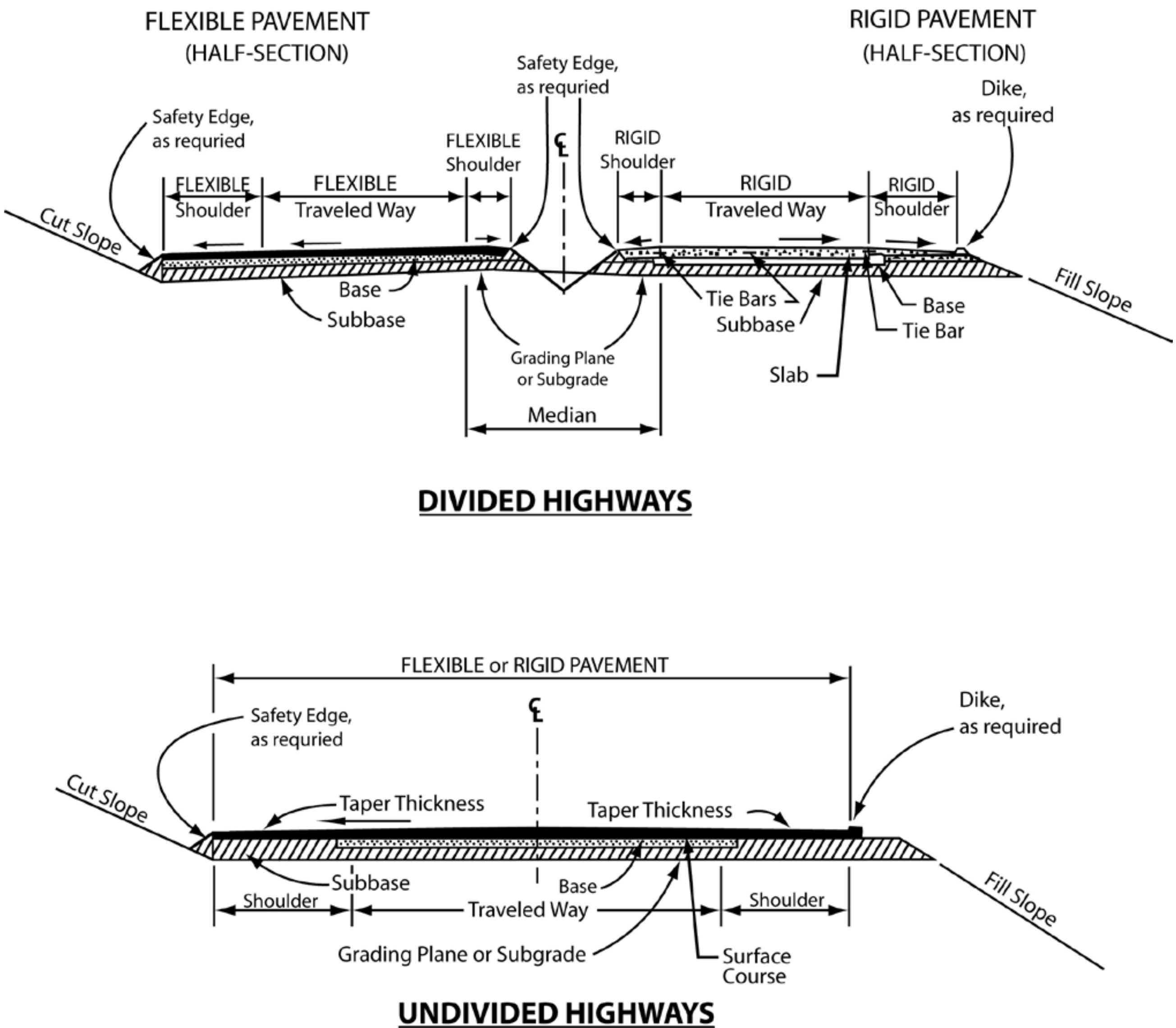
Pavement Preservation has two main categories or programs:

(1) *Preventive Maintenance.* Preventive maintenance projects are used to provide preventive treatments to preserve pavements in good condition. These projects are typically done by Department Maintenance forces or through the Major Maintenance Program. The District Maintenance Engineer typically determines which preventive treatment to apply and when. Examples of preventive maintenance projects include:

- Removal and replacement of a non-structural wearing course (for example, open graded friction courses);
- Thin non-structural overlays less than or equal to 0.08 foot (or 0.10 foot when needed to enhance compaction in colder temperatures);

Figure 602.1

Basic Pavement Layers of the Roadway



NOTES:

1. These illustrations are only to show nomenclature and are not to be used for geometric cross section details. For these, see Chapter 300.
2. Pavement drainage design, both on divided and undivided highways, are illustrated and discussed under Chapter 650.
3. Only flexible and rigid pavements shown. Composite pavements are the same as rigid pavements with a flexible layer overlay.
4. See Index 626.2 for criteria for when and how to use flexible or rigid shoulders.

- Replacing joint seals; crack sealing; grinding or grooving rigid pavement surface to improve friction;
- Grinding rigid pavement to eliminate rutting from chain wear;
- Seal coats; slurry seals; and microsurfacing.

Traffic safety and other operational improvements, geometric upgrades, or widening are normally not included in preventative maintenance projects. Strategies and guidelines on preventive maintenance treatments currently used by the Department are available in the Maintenance Policy Directive. Note that such strategies are periodically updated.

(2) *Capital Preventive Maintenance (CAPM).* Capital Preventive Maintenance (CAPM) is a program of short-term (5 to less than 20 years) repair projects agreed to between the Department and FHWA in 1994. Detailed information regarding the CAPM program can be found in Design Information Bulletin 81. CAPM Guidelines available on the Department Pavement website and in Chapters 620, 630 and 640 of this manual.

The primary purpose of the CAPM program is to repair pavement exhibiting minor surface distress and/or triggered ride (International Roughness Index (IRI) greater than 170 inches per mile) as determined by the Pavement Condition Survey (PCS) and the Pavement Management System (PMS). Ride improvement and preservation of serviceability are key elements of this program. Timely application of CAPM treatments will postpone the need for major roadway rehabilitation and is generally more cost effective than having to rehabilitate pavements exhibiting major distress. CAPM gives the districts the flexibility to make the most effective use of all funds available in the biennial State Highway Operation and Protection Plan (SHOPP).

Since the CAPM program is part of pavement preservation, CAPM projects are more closely related to preventive maintenance (Major Maintenance) projects than to roadway rehabilitation projects. CAPM projects involve non-structural overlays and repairs, which do not require Traffic Index calculations or deflection studies. CAPM projects include all appropriate items or work necessary to construct and address impacts from the pavement. See DIB 82 for required work regarding accessibility for persons with disabilities. Limited drainage and traffic operational work can also be included when appropriate, but they do not include major facility upgrades like widening, geometric upgrades, or roadside upgrades. Further information on CAPM strategies, including appropriate drainage/operational work and other guidance for CAPM projects, can be found in the CAPM Guidelines.

Examples of CAPM projects include:

- Surface course overlays less than or equal to 0.20 foot (0.25 foot if International Roughness Index >170 in/mile).
- Removal and replacement of surface course (not to exceed the depth of the surface course overlay).
- Surface in-place recycling projects. (Overlay to not exceed 0.20 foot for Hot Mix Asphalt and 0.15 foot for Rubberized Hot Mix Asphalt.)
- Individual rigid pavement slab replacements or punchout repairs.
- Diamond grinding of rigid pavements to eliminate faulting or restore ride quality to an acceptable level.
- Dowel bar retrofit.

Items that are not considered CAPM include:

- Crack, seat, and overlay of rigid pavements.
- Surface course overlays greater than 0.25 foot.
- Removal and replacement of more than 0.25 foot of the surface course (unless the work is incidental to maintaining an existing vertical clearance or to conform to existing bridges or pavements).

- Lane/shoulder replacements (including pulverization and other base restoration/recycling projects).

Projects that require these types of treatments are roadway rehabilitation projects and should meet those standards, see Index 603.4.

603.4 Roadway Rehabilitation

The primary purpose of roadway rehabilitation projects is to return roadways that exhibit major structural distress, to good condition. Many of these structural distresses indicate failure of the surface course and underlying base layers. Roadway rehabilitation work is generally regarded as major, non-routine maintenance work engineered to preserve and extend the service life as well as provide upgrades to enhance safety where needed. As described in Design Information Bulletin 79, Section 1.2, rehabilitation criteria also apply to minor projects and certain other projects in addition to roadway rehabilitation projects. Roadway rehabilitation is different from pavement preservation that simply preserves or repairs the facility to a good condition.

Roadway rehabilitation projects are divided into 2R (Resurfacing and Restoration) and 3R (Resurfacing, Restoration and Rehabilitation). Roadway rehabilitation projects should address other highway appurtenances such as pedestrian and bicyclist facilities, drainage facilities lighting, signal controllers, and fencing that are failing, worn out or functionally obsolete. Also, unlike pavement preservation projects, geometric enhancements and operational improvements may be added to roadway rehabilitation work if such work is critical or required by FHWA standards. Where conditions warrant, quieter pavement strategies could be used to reduce tire/pavement noise. In certain cases, where traditional noise abatement is infeasible, quieter pavement strategies may be considered as an alternative. See Chapter 1100 for additional information on highway traffic noise abatement.

Examples of roadway rehabilitation projects include:

- Overlay.
- Removal and replacement of the surface course.
- Crack, seat, and overlay of rigid pavements regardless of overlay thickness.
- Lane/shoulder replacements.

Roadway rehabilitation strategies for rigid, flexible and composite pavements are discussed in Chapters 620, 630 and 640. Additional information and guidance on roadway rehabilitation, including determining whether the project fits 2R or 3R screening criteria, and other rehabilitation projects may also be found in the Design Information Bulletin, Number 79 - "Design Guidance and Standards for Roadway Rehabilitation Projects" and in the PDPM Chapter 9, Article 5.

603.5 Reconstruction

Pavement reconstruction is the replacement of the entire existing pavement structure by the placement of the equivalent or increased pavement structure. Reconstruction usually requires the complete removal and replacement of the existing pavement structure utilizing either new or recycled materials. Reconstruction is required when a pavement has either failed or has become structurally or functionally outdated.

Reconstruction features typically include the addition of lanes, as well as significant change to the horizontal or vertical alignment of the highway. Although reconstruction is often done for other reasons than pavement repair (realignment, vertical curve correction, improve vertical clearance, etc.), it can be done as an option to rehabilitation when the existing pavement meets any of the following conditions:

- Is in a substantially distressed condition and rehabilitation strategies will not restore the pavement to a good condition; or
- Grade restrictions prevent overlaying the pavement to meet the pavement design life requirements for a rehabilitation project; or
- Life cycle costs for rehabilitation are greater than those for reconstruction.

Reconstruction differs from lane/shoulder replacement roadway rehabilitation options in that lane/shoulder replacements typically involve replacing isolated portions of the roadway width whereas reconstruction is the removal and replacement of the entire roadway width. Incidental rebuilding of existing pavements for rehabilitation in order to conform to bridges, existing pavement, or meet vertical clearance standards are also considered a rehabilitation and not reconstruction. Storm or earthquake damage repair (i.e., catastrophic) also are not considered reconstruction projects.

Pavement reconstruction projects are to follow the same standards as new construction found in this manual unless noted otherwise.

603.6 Temporary Pavements and Detours

Temporary pavements and detours are constructed to temporarily carry traffic anticipated during construction. These types of pavements should be engineered using the standards and procedures for new construction except where noted otherwise.

Topic 604 - Roles and Responsibilities

604.1 Roles and Responsibilities for Pavement Engineering

The roles and responsibilities listed below apply only to pavement engineering.

(1) *Pavement Engineer*. The pavement engineer is the engineer who performs pavement calculations, develops pavement structure recommendations, details, or plans. The pavement engineer can be the Project Engineer, District Materials Engineer, District Maintenance Engineer, consultant, or other staff engineer responsible for this task.

(2) *Project Engineer (PE)*. The PE is the registered civil engineer in responsible charge of appropriate project development documents (i.e., Project Study Report, Project Report, and PS&E) and coordinates all aspects of project development. The PE is responsible for project technical decisions, engineering quality (quality control), and estimates. This includes collaborating with the District Materials Engineer, District Pavement Advisor and other subject matter experts regarding pavement details and selecting pavement strategy for new and rehabilitation projects. The PE clearly conveys pavement related decisions and information on the project plans and specifications for a Contractor to bid and build the project.

The PE coordinates with the Structures District Liaison Engineer and Division of Engineering Services (DES) staff for the proper selection and engineering of any structure approach system including the adequacy of all drainage ties between the structure approach drainage features and other new or existing drainage facilities. The PE should contact the Structures District Liaison Engineer as early as possible in the project development process to facilitate timely review and project scheduling.

(3) *District Materials Engineer (DME).* The DME is responsible for materials information for pavement projects in the district. The District Materials Unit is responsible for conducting or reviewing the findings of a preliminary soils and other materials investigation to evaluate the quality of the materials available for constructing the project. The DME prepares or reviews the Materials Report for each project; provides recommendations to and in continuous consultation with the Project Engineer throughout planning and design, and with the PE and Resident Engineer during construction; and coordinates Materials information with the Department functional units, Material Engineering and Testing Services (METS), Headquarters functional units, local agencies, industry, and consultants.

(4) *District Pavement Advisor (DPA).* The DPA manages and coordinates overall pavement strategies for the District. They are primarily involved in pavement management such as identifying future pavement preservation, rehabilitation, and reconstruction needs, and prioritizing pavement projects to meet those needs. The DPA establishes pavement projects and reviews planning documents prepared by the PE for consistency with overall District and statewide goals for pavements. The District Pavement Advisor is typically either the District Maintenance Engineer or another individual within District Maintenance.

(5) *Pavement Program (PP)*. The PP, within the Division of Maintenance (DOM) is responsible for statewide standards and guidelines for the pavement engineering process. The DOM Assistant Division Chief for Pavement Program serves as the State Pavement Engineer for the Department.

The PP Office of Concrete Pavement and Pavement Foundations (OCPPF) and Asphalt Pavement (OAP) are responsible for maintaining pavement engineering standards, specifications, standard plans, design methodologies, design software, and practices that are used state wide. OCPPF and OAP also provide technical expertise on material properties and products for pavements. OCPPF and OAP work closely with the District Materials Engineers, Maintenance Engineers, and Resident Engineers to investigate ongoing field and materials issues.

(6) *State Pavement Engineer.* The State Pavement Engineer provides leadership and commitment to ensure safe, effective, and environmentally sensitive highway pavements that improve mobility across California. The State Pavement Engineer is responsible for conveying clear direction and priorities on pavement initiatives, policies, and standards that reflect departmental goals; and for the implementation of pavement policies, standards, and specifications.

(7) *Division of Engineering Services (DES).* The following units within DES provide services that relate to pavements:

- *Materials and Geotechnical Services:* The Materials and Geotechnical Services subdivision consists of the Materials unit (formerly Materials Engineering and Testing Services (METS)) and the Geotechnical Services (GS) unit. The Materials unit is responsible for conducting laboratory testing, field testing, specialized field inspections, and maintaining the test method procedures forthe Department. The GS unit provides the Districts, Structures, and Headquarters with expertise and guidance in soil related investigations and groundwater issues, GS prepares or reviews Geotechnical Design Reports based upon studies and information supplied by the District.

- *Structure Design (SD):* Structure Design is responsible for selecting the type of structure approach system to be used when the construction or rehabilitation of a structure approach slab is necessary.

604.2 Other Resources

The following resources provide additional standards and guidance related to pavement engineering. Much of this information can be found on the Department Pavement website, see category (5) below.

(1) *Standard Plans*. These are collections of commonly used engineering details intended to provide consistency for contractors, resident engineers and maintenance engineers in defining the scope of work for projects, assist in the biddability of the project contract plans, and assist maintenance in maintaining the facility. The standard plans were developed based on research and field experience and in consultation with industry. Standard plans for pavement should not be altered or modified without the prior written approval of the Chief, Office of Concrete Pavement and Pavement Foundations. Standard plans for pavements can be found on the Department Pavement website.

(2) *Standard Specifications and Standard Special Provisions.* The Standard Specifications provide material descriptions, properties and work quality requirements, contract administration requirements, and measurement and payment clauses for items used in the project. The Standard Special Provisions are additional specification standards used to modify the Standard Specifications including descriptions, quality requirements, and measurement and payment for the project work and materials. When no Standard Specification or Standard Special Provision exists for new or proprietary items, the Pavement Program must review and concur with a special provision. For further information, see the Specifications section on the Department Pavement website.

(3) *Pavement Technical Guidance.* Pavement Technical Guidance is a collection of supplemental guidance and manuals regarding pavement engineering which is intended to assist project engineers, pavement engineers, materials engineers, consultants, construction oversight personnel, and maintenance workers in making informed decisions on pavement structural engineering, constructability and maintainability issues. Information includes, but is not limited to, resources for assistance in decision making, rigid, flexible and composite pavement rehabilitation strategies, pavement preservation strategies, and guidelines for the use of various products and materials. Technical assistance is also available from the Pavement Program to assist with pavements that utilize new materials, methods, and products. These Technical Guidance documents may be accessed on the Department Pavement website.

(4) *Supplemental District Standards and Guidance.* Some Districts have developed additional pavement standards and guidance to address local issues. Such guidance adds to or supplements the standards found in this manual, the Standard Plans, the Standard Specifications, and Standard Special Provisions. District guidance does not replace minimum statewide standards unless the State Pavement Engineer has approved an exception. Supplemental District Guidance can be obtained by contacting the District Materials Engineer.

(5) *Department Pavement website.* The Department Pavement website provides a one-stop resource for those seeking to find standards, guidance, reports, approved software, and other resource tools related to pavements. The Department Pavement website can be accessed at http://www.dot.ca.gov/hq/esc/Translab/OPD/DivisionofDesign-Pavement-Program.htm.

(6) *Pavement Interactive Guide*. The Pavement Interactive Guide is a reference tool developed by the Department in partnership with other states. It includes discussion and definitions to terms and practices used in pavement engineering that are intended to aid design engineers in obtaining a better understanding of pavements. This document is not a standards manual or guideline, rather, it supplements the standards, definitions, and guidance in this manual. Because of copyright issues, the Pavement Interactive Guide is only available to Department employees on the Pavement intranet, or internal, website.

(7) *The AASHTO "Guide for Design of Pavement Structures*. Although not adopted by the Department, the AASHTO "Guide for Design of Pavement Structures" is a comprehensive reference guide that provides background that is helpful to those involved in engineering of pavement structures. This reference is on file in the Pavement Program and a copy should be available in each District. Engineering procedures included in the AASHTO Guide are used by FHWA to check the adequacy of the specific pavement structures adopted for the Department projects, as well as the procedures and standards included in Chapters 600 - 670 of this manual.

Topic 605 – Record Keeping

605.1 Documentation

One complete copy of the documentation for the type of pavement selected should be retained in permanent District Project History files as well as subsequent updates of construction changes to the pavement structure. The documentation must contain the following:

- Pavement design life (including both the construction year and design year),
- The California R-values and unified soil classification of the subgrade soil,
- The California R-value(s) or strength properties for the materials selected for the subbase and/or base layers,
- The Traffic Index (TI) for each pavement structure, and
- Life cycle cost analysis (including the data required for the life-cycle cost analysis) and other factors mentioned in Topic 619.

605.2 Subsequent Revisions

Any subsequent changes in pavement structures must be documented and processed in accordance with the appropriate instructions stated above and with proper reference to the original design.

Topic 606 - Research and Special Designs

606.1 Research and Experimentation

Research and experimentation are undertaken on an ongoing basis to provide improved methods and standards, which take advantage of new technology, materials, and practices. They may involve investigations of new materials, construction methods, and/or new engineering procedures. Submittal of new ideas by Headquarters and District staff, especially those involved in the engineering, construction, maintenance, paving materials, and performance of the pavement, is encouraged. Research proposals should be sent to the Division of Research and Innovation in Headquarters for review and consideration. Suggestions for research studies and changes in pavement standards may also be submitted to the State Pavement Engineer. The Pavement Program must approve pilot projects and experimental construction features before undertaking such projects. District Maintenance should also be engaged in the discussion involving pilot projects and experimental construction features. Experimental sections must be clearly marked so that District Maintenance can easily locate and maintain such sites.

606.2 Special Designs

Special designs must be fully justified and submitted to the Headquarters Pavement Program, Office of Concrete Pavement and Pavement Foundations (OCPPF) for approval. "Special" designs defined as those designs that meet either or both of the following criteria:

- Involve products, methods, or strategies which either reduce the structural thickness to less than what is determined by the standards and procedures of this manual and accompanying technical guidance, or
- Utilize experimental products or procedures (such as mechanistic-empirical engineering method) not covered in the engineering tables or methods found in this manual or accompanying technical guidance.

Special designs must be submitted to the Headquarters Pavement Program, Office of Concrete Pavement and Pavement Foundations (OCPPF) either electronically or as hard copies. Hard copy submittals must be in duplicate. All submittals must include the proposed pavement structure(s) and a location strip map (project title sheet is acceptable). The letter of transmittal should include the following:

- Pavement design life, including both the construction year and design year (See Topic 612).
- The California R-value(s) and unified soil classification of the subgrade soil(s) (See Indexes 614.2 and 614.3).
- The California R-value(s) or strength properties for the materials selected for the subbase and/or base layers (See Tables 663.1A and 663.1B).
- The Traffic Index (TI) for each pavement structure (See Indexes 613.3 & 613.4).
- Justification for the "special" design(s).

OCPPF will act as the Headquarters focal point to obtain concurrence of Pavement Program and other Headquarters functional units as needed prior to OCPPF granting approval of the "special" designs.

606.3 Mechanistic-Empirical Design

Mechanistic-Empirical (ME) Design is currently under development by the Department, FHWA, AASHTO and other States. On March 10, 2005, the Department committed to develop ME Design as an alternative and possible replacement of current methods. The Department is currently working on the procedures and criteria for performing this analysis. Until the criteria are established and the methodology verified, ME Design will be considered experimental and cannot, at this time, be used to engineer pavements on the State highway system or other roads maintained by the State.

606.4 Proprietary Items

The use of proprietary materials and methods on State highway projects is discussed in Topic 110.10.

CHAPTER 610
PAVEMENT ENGINEERING CONSIDERATIONS

Topic 611 - Factors In Selecting Pavement Type

Index 611.1 Pavement Type Selection

The types of pavement generally considered for new construction and rehabilitation in California are rigid, flexible and composite pavements. Rigid pavement should be considered as a potential alternative for all Interstate and other high traffic volume interregional freeways. Flexible pavement should be considered as a potential alternative for all other State highway facilities. Composite pavement, which consists of a flexible layer over a rigid pavement have mostly been used for maintenance and rehabilitation of rigid pavements on State highway facilities.

611.2 Selection Criteria

Because physical conditions and other factors considered in selecting pavement type vary significantly from location to location, the Project Engineer must evaluate each project individually to determine the most appropriate and cost-effective pavement type to be used. The evaluation should be based on good engineering judgment utilizing the best information available during the planning and design phases of the project together with a systematic consideration of the following project specific conditions:

- Pavement design life
- Traffic considerations
- Soils characteristics
- Weather (climate zones)
- Existing pavement type and condition
- Availability of materials
- Recycling
- Maintainability
- Constructibility
- Cost comparisons (initial and life-cycle)

The above factors should be thoroughly investigated when selecting a pavement structure and addressed specifically in all project documents (PSSR, PSR, PR, PS&E, etc). The final decision on pavement type should be the most economical design based on life-cycle cost analysis (see Topic 619.)

The principal factors considered in selecting pavement structures are discussed as follows in Topics 612 through 619.

Topic 612 - Pavement Design Life

612.1 Definition

Pavement design life, also referred to as performance period, 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 traffic volume and loading. The strategy or pavement structure selected for any project should provide the minimum pavement design life that meets or exceeds the objective of the project as described in Topics 612 through 619.

612.2 New Construction and Reconstruction

The minimum pavement design life for new construction and reconstruction projects shall be no less than the values in Table 612.2 or the project design period (see Index 103.2), whichever is greater.

612.3 Widening

Additional consideration is needed when determining the design life for pavement widening. Factors to consider include the remaining service life of the adjacent pavement, planned future projects (including maintenance and rehabilitation), and future corridor plans for any additional lane widening and shoulders. **The pavement design life for widening projects shall either match the remaining pavement service**

Table 612.2

Pavement Design Life for New Construction and Reconstruction

Facility	Pavement Design Life (Years)	
	AADT[3] <150,000[1] and AADTT[4] <15,000[1]	AADT ≥ 150,000[1] or AADTT ≥ 15,000[1]
Mainline Traveled Way	20 or 40 [2]	40
Ramp Traveled Way	20 or 40 [2]	40
Shoulders:		
≤5 ft wide	Match adjacent traveled way	40
>5 ft wide: First 2 ft	Match adjacent traveled way	40
Remaining width [5]	20	20
Intersections	20 or 40 [2]	40
Roadside Facilities	20	20

NOTES:

(1) Projected mainline AADT and AADTT in both directions, 20 years after construction

(2) Use design life with lowest life-cycle cost (See Topic 619)

(3) Annual Average Daily Traffic (AADT)

(4) Annual Average Daily Truck Traffic (AADTT)

(5) If the shoulder is expected to be converted to a traffic lane with the pavement design life, it should be engineered to match the same pavement design life as the adjacent traveled way.

life of the adjacent roadway (but not less than the project design period as defined in Index 103.2), or the pavement design life values in Table 612.2 depending on which has the lowest life-cycle costs. Life-cycle cost analysis is discussed further in Topic 619.

When widening a roadway, the existing pavement should be rehabilitated and brought up to the same life expectancy as the new widened portion of the roadway.

612.4 Pavement Preservation

(1) *Preventive Maintenance:* Because preventive maintenance projects involve non-structural overlays, seals, grinds, or repairs, they are not engineered to meet a minimum structural design life like other types of pavement projects. Their intended goal is to extend the service life of an existing pavement structure while it is in good condition. Typically, for preventive maintenance, the added service life can vary from a couple of years to over 7 years depending on the strategy being used and the condition of the existing pavement.

(2) *Capital Preventive Maintenance:* The strategies used for CAPM projects have been engineered to extend the service life of a pavement that exhibits minor distress and/or triggered ride (International Roughness Index (IRI) greater than 170 inches per mile) by a minimum of 5 years. Some strategies such as rigid pavement diamond grinding, slab replacement, punchout repairs, and dowel bar retrofit can last at least 10 years.

612.5 Roadway Rehabilitation

The minimum pavement design life for roadway rehabilitation projects shall be 20 years except for roadways with existing rigid pavements or with a current Annual Average Daily Traffic (AADT) of at least 15,000 vehicles, where the minimum pavement design life shall be 20 or 40 years depending on which design life has the lowest life-cycle costs. At the discretion of the District, a 40-year pavement design life may be considered and evaluated for all projects with an AADT less than 15,000 using the Department's life cycle cost analysis procedures. Life-cycle cost analysis is discussed further in Topic 619.

612.6 Temporary Pavements and Detours

<u>Temporary pavements and detours should be engineered to accommodate the anticipated traffic loading that the pavement will experience during the construction period.</u> The minimum design life for temporary pavements and detours should be no less than the construction period for the project. This period may range from a few months to several years depending on the type, size and complexity of the project.

612.7 Non-Structural Wearing Courses

As described in Index 602.1(5), a non-structural wearing course is used on some pavements to ensure that the underlying layers will be protected from wear and tear from tire/pavement interaction, the weather, and other environmental factors for the intended design life of the pavement. Because non-structural wearing courses are not considered to contribute to pavement structural capacity, they are not expected to meet the same design life criteria as the structural layers. However, when selecting materials, mix designs and thickness of these courses, appropriate evaluation and sound engineering judgment should be used to optimize performance and minimize the need for maintenance of the wearing course and the underlying structural layers. Based on experience, a properly engineered non-structural wearing course placed on new pavement should perform adequately for 10 or more years, and 5 or more years when placed on existing pavement as a part of rehabilitation or preventive maintenance.

Topic 613 - Traffic Considerations

613.1 Overview

Pavements are engineered to carry the truck traffic loads expected during the pavement design life. Truck traffic, which includes transit vehicles trucks and truck-trailers, is the primary factor affecting pavement design life and its serviceability. Passenger cars and pickups are considered to have negligible effect when determining traffic loads.

Truck traffic information that is currently required for pavement engineering includes projected volume for each of four categories of truck and transit vehicle types by axle classification (2-, 3-, 4-, and 5-axles or more). When the Department adopts the Mechanistic – Empirical (ME) design method, additional information such as axle configurations (single, tandem, tridem, and quad), axle loads, and number of load repetitions would also be required. This information is used to estimate anticipated traffic loading and performance of the pavement structure. The Department currently estimates traffic loading by using established constants for a 10-, 20-, 30-, or 40-year pavement design life to convert truck traffic data into 18-kip equivalent single axle loads (ESALs). The total projected ESALs during the pavement design life are in turn converted into a Traffic Index (TI) that is used to determine minimum pavement thickness. Another method for estimating pavement loading known as Axle Load Spectra is currently under development by the Department for future use with the Mechanistic-Empirical (ME) design procedure.

613.2 Traffic Volume Projections

(1) *Traffic Volume and Loading Data.* In order to determine expected traffic loads on a pavement it is first necessary to determine projected traffic volumes during the design life for the facility.

Traffic volume or loading on State highways can come from vehicle counts and classification, weigh-in-motion (WIM) stations, or the Truck Traffic (Annual Average Daily Truck Traffic) on California State Highways published annually by Headquarters Division of Traffic Operations. Current and projected traffic volume by vehicle classification must be obtained for each project in accordance with the procedures found in this Topic.

Districts typically have established a unit within Traffic Operations or Planning specifically responsible for providing travel forecast information. These units are responsible for developing traffic projections (including truck volumes, equivalent single axle loads, and TIs) used for planning and engineering of State highways in the District. The Project Engineer should coordinate with the forecasting unit in their District early in the project development process to obtain the required traffic projections.

(2) *Design Year Annual Average Daily Truck Traffic (AADTT):* An expansion factor obtained from the traffic forecasting unit is used to project current AADTT to the design year AADTT for each axle classification (see Table 613.3A). In its simplest form, the expansion factor is a straight-line projection of the current one-way AADTT data. When using the straight-line projection, the truck traffic data is projected to find the AADTT at the midway of the design life. This represents the average one-way AADTT for each axle classification during the pavement design life.

When other than a straight-line projection of current truck traffic data is used for engineering purposes, the procedure to be followed in developing design year traffic projections will depend on travel forecast information for the region. In such cases, the projections require a coordinated effort from the District's Division of Transportation Planning and Traffic Operations, working closely with the Regional Agencies to establish realistic values for truck traffic growth rates based on travel patterns, land use changes, and other socioeconomic factors.

Due to various changes in travel patterns, land use changes, and other socioeconomic factors that may significantly affect design year traffic projections, the TI for facilities with longer service life, such as a 30- or 40-year design life require more effort to determine than for a 10- or 20-year design life. For this reason, the Project Engineer should involve District Transportation Planning and/or Traffic Operations in determining a realistic and appropriate TI for each project early in the project development process. In the absence of 30- or 40-year traffic projection data, 20-year projection data may be extrapolated to

30- and 40-year values by applying the expansion factors.

613.3 Traffic Index Calculation

The Traffic Index (TI) is determined using the following procedures:

(1) *Determine the Projected Equivalent Single Axle Loads (ESALs)*. The information obtained from traffic projections and Truck Weight Studies is used to develop 18-kip Equivalent Single Axle Load (ESAL) constants that represent the estimated total accumulated traffic loading for each heavy vehicle (trucks and buses and each of the four truck types during the pavement design life. Typically, buses are assumed to be included in the truck counts due to their relatively low number in comparison to trucks. However, for facilities with high percentage of buses such as high-occupancy vehicle (HOV) lanes and exclusive bus-only lanes, projected bus volumes need to be included in the projection used to determine ESALs. The ESAL constants are used as multipliers of the projected AADTT for each truck type to determine the total cumulative ESALs and in turn the Traffic Index (TI) during the design life for the pavement (see Index 613.3(3)). The ESALs and the resulting TI are the same magnitude for both flexible, rigid, and composite pavement alternatives. The current 10-, 20-, 30-, and 40-year ESAL constants are shown in Table 613.3A.

(2) *Lane Distribution Factors.* Truck/bus traffic on multilane highways normally varies by lane with the lightest volumes generally in the median lanes and heaviest volumes in the outside lanes. Buses are also typically found in HOV lanes. For this reason, the distribution of truck/bus traffic by lanes must be considered in the engineering for all multilane facilities to ensure that traffic loads are appropriately distributed. Because of the uncertainties and the variability of lane distribution of trucks on multilane freeways and expressways, statewide lane distribution factors have been established for pavement engineering of highway facilities in California.

These lane distribution factors are shown in Table 613.3B.

(3) *Traffic Index (TI).* The Traffic Index (TI) is a measure of the number of ESALs expected in the traffic lane over the pavement design life of the facility. The TI does not vary linearly with the ESALs but rather according to the following exponential formula and the values presented in Table 613.3C. The TI is determined to the nearest 0.5.

$$TI = 9.0 \times \left(\frac{(ESAL \times LDF)}{10^6} \right)^{0.119}$$

Where:

TI = Traffic Index

ESAL = Total number of cumulative 18-kip Equivalent Single Axle Loads

LDF = Lane Distribution Factor (see Table 613.3B)

Index 613.4 contains additional requirements and considerations for determining projected traffic loads.

613.4 Axle Load Spectra

(1) *Development of Axle Load Spectra.* Axle load spectra is an alternative method of measuring heavy vehicle loads that is currently under development for the future mechanistic-empirical design method. Axle load spectra is a representation of normalized axle load distribution developed from weigh-in-motion (WIM) data for each axle type (single, tandem, tridem, and quad) and truck class (FHWA vehicle classes 4 through 13). Axle load spectra do not involve conversion of projected traffic loads into equivalent single axle loads (ESALs), instead traffic load applications for each truck class and axle type are directly characterized by the number of axles within each axle load range.

In order to accurately predict traffic load related damage on a pavement structure, it is important to develop both spatial and temporal axle load spectra for different truck loadings and pavements. The following data is needed to develop axle load spectra:

Table 613.3A
ESAL Constants

Vehicle Type (By Axle Classification)	10-Year Constants	20-Year Constants	30-Year Constants	40-Year Constants
2-axle trucks or buses	690	1,380	2,070	2,760
3-axle trucks or buses	1,840	3,680	5,520	7,360
4-axle trucks	2,940	5,880	8,820	11,760
5 or more-axle trucks	6,890	13,780	20,670	27,560

Table 613.3B
Lane Distribution Factors for Multilane Highways

Number of Mixed Flow Lanes in One Direction	Factors to be Applied to Projected Annual Average Daily Truck Traffic (AADTT)			
	Mixed Flow Lanes (see Notes 1-6)			
	Lane 1	Lane 2	Lane 3	Lane 4
One	1.0	-	-	-
Two	1.0	1.0	-	-
Three	0.2	0.8	0.8	-
Four	0.2	0.2	0.8	0.8

NOTES:
1. Lane 1 is next to the centerline or median.
2. For more than four lanes in one direction, use a factor of 0.8 for the outer two lanes plus any auxiliary/collector lanes, use a factor of 0.2 for other mixed flow through lanes.
3. For HOV lanes and other inside lanes (non truck lanes), use a factor of 0.2. However, as noted in Index 613.5(1)(b), the TI should not be less than 10 for a 20-year pavement design life, or than 11 for a 40-year pavement design life. Additionally, for freeways and expressways, the maximum TI must not exceed 11 or 12 for a 20-year and 40-year design life, respectively.
4. If trucks are permitted to use HOV or other inside lanes, HOV and/or other inside lanes shall be designed to the same standards as found in this table for the outside lanes.
5. For lanes devoted exclusively to buses and/or trucks, use a factor of 1.0 based on projected AADTT of mixed-flow lanes for auxiliary and truck lanes, and a separate AADTT based on expected bus traffic for exclusive bus-only lanes.
6. The lane distribution factors in this table represent minimum factors and, based on knowledge of local traffic conditions and sound engineering judgment, higher values should be used for specific locations when warranted.

Table 613.3C
Conversion of ESAL to Traffic Index

ESAL [(1)]	TI [(2)]	ESAL [(1)]	TI [(2)]
4,710		6,600,000	
	5.0		11.5
10,900		9,490,000	
	5.5		12.0
23,500		13,500,000	
	6.0		12.5
47,300		18,900,000	
	6.5		13.0
89,800		26,100,000	
	7.0		13.5
164,000		35,600,000	
	7.5		14.0
288,000		48,100,000	
	8.0		14.5
487,000		64,300,000	
	8.5		15.0
798,000		84,700,000	
	9.0		15.5
1,270,000		112,000,000	
	9.5		16.0
1,980,000		144,000,000	
	10.0		16.5
3,020,000		186,000,000	
	10.5		17.0
4,500,000		238,000,000	
	11.0		17.5 [(3)]
6,600,000		303,000,000	

Notes:

(1) For ESALs less than 5,000 or greater than 300,000,000, use the TI equation to calculate design TI, see Index 613.3(3).

(2) The determination of the TI closer than 0.5 is not justified. No interpolations should be made.

(3) For TI's greater than 17.5, use the TI equation, see Index 613.3(3).

- Truck class (FHWA Class 4 for buses through Class 13 for 7+ axle multi-trailer combinations)
- Axle type (single, tandem, tridem, and quad)
- Axle load range for each axle type and truck class (3 to 102 kips)
- The number of axle load applications within each axle load range by axle type and truck class
- The percentage of the total number of axle applications within each axle load range with respect to each axle type, truck class, and year of data. These are the normalized values of axle load applications for each axle type and truck class

The aforementioned data are obtained from traffic volume counts and WIM data for vehicle classification, and axle type and weight. Traffic counts and WIM stations should be deployed widely to ensure that projected volume estimates for each vehicle class and axle type are in line with the actual volumes and growth rates.

(2) *Use of Axle Load Spectra in Pavement Engineering:* Pavement engineering calculations using axle load spectra are generally more complex than those using ESALs or Traffic Index (TI) because loading cannot be reduced to one equivalent number. However, the load spectra approach of quantifying traffic loads offers a more practical and realistic representation of traffic loading than using TI or ESALs. Due to its better performance modeling, axle load spectra will be used in the Mechanistic-Empirical (M-E) design method currently under development to evaluate traffic loading over the design life for new and rehabilitated pavements. This information will be used to validate original pavement design loading assumptions, and to continuously monitor pavement performance given the loading spectrum. Axle load spectral data will also be used to facilitate effective and pro-active deployment of maintenance efforts and in the development of appropriate strategies to mitigate sudden and unexpected pavement deterioration due to increased volumes or loading patterns.

In this edition of the Highway Design Manual, axle load spectra are not used to engineer pavements.

613.5 Specific Traffic Loading Considerations

(1) *Traveled Way.*

(a) Mainline Lanes. Because each lane for a multilane highway with 3 or more lanes in each direction may have a different load distribution factor (see Table 613.3B), multiple TIs may be generated for the mainline lanes which can result in different pavement thickness for each lane. Such a design with different thickness for each lane would create complications for constructing the pavement. Therefore, the decision to use a single or multiple TI’s for the pavement engineering of mainline lanes for a multilane highway with 3 or more lanes in each direction should be based on a thorough consideration of constructibility issues discussed in Index 618.2 together with sound engineering judgment. If one TI is used, it should be the one that produces the most conservative pavement structure.

(b) Freeway Lanes. TI for new freeway lanes, including widening, auxiliary lanes, and high-occupancy vehicle (HOV) lanes, should be the greater of either the calculated value, 10.0 for a 20-year pavement design life, or 11.0 for a 40-year pavement design life. For roadway rehabilitation projects, use the calculated TI.

(c) Ramps and Connectors:

1. Connectors. AADTT and TI's for freeway-to-freeway connectors should be determined the same way as for mainline traffic.

2. Ramps to Weigh Stations. Pavement structure for ramps to weigh stations should be engineered using the mainline ESALs and the load distribution factor of 1.0 for exclusive truck lanes as noted in Table 613.3B.

3. Other Ramps. Estimating future truck traffic on ramps is more difficult than on through traffic lanes. It is typically more difficult to accurately forecast ramp AADTT because of a much greater impact of commercial and industrial development on ramp truck traffic than it is on mainline truck traffic.

   If reliable truck traffic forecasts are not available, ramps should be engineered using the 10-, 20-, and 40-year TI values given in Table 613.5A for light, medium, and heavy truck traffic ramp classifications. Design life TI should be the greater of the calculated TI or the TI values in Table 613.5A.

The three ramp classifications are defined as follows:

- Light Traffic Ramps - Ramps serving undeveloped or residential areas with light to no truck traffic predicted during the pavement design life.
- Medium Traffic Ramps - Ramps in metropolitan areas, business districts, or where increased truck traffic is likely to develop because of anticipated commercial development within the pavement design life
- Heavy Traffic Ramps - Ramps that will or currently serve industrial areas, truck terminals, truck stops, and/or maritime shipping facilities.

The final decision on ramp truck traffic classification rests with the District.

Table 613.5A
Traffic Index (TI) Values for Ramps and Connectors

Ramp Truck Traffic Classification	Minimum Traffic Index (TI)	
	20–Yr Design Life	40–Yr Design Life (1)
Light	8.0	9.0
Medium	10.0	11.0
Heavy	12.0	14.0

NOTE:

(1) Based on straight line extrapolation of 20-year ESALs.

(2) Shoulders.

(a) Purpose and Objectives.

Shoulder pavement structures must be designed and constructed to assure that the following performance objectives are met:

- Be safely and economically maintained.
- Enhance the performance of adjacent travel lanes.
- Be structurally adequate to handle maintenance and emergency vehicles and to serve as emergency parking.
- Accommodate pedestrians and bicyclists as necessary.
- Provide versatility in using the shoulders as temporary detours for construction or maintenance activities in the future.
- Make it easier and more cost-effective to convert into a traffic lane as part of a future widening.
- Simplify the Contractor’s operation which leads to reduced working days and lower unit prices.

Shoulders do not need to be designed to traffic lane standards to meet these objectives. To achieve these performance objectives, the following design standards apply for shoulders on the State highway.

(b) New Construction and Reconstruction. **New or reconstructed shoulders shall be engineered to match the TI of the adjacent traffic lane when any of the following conditions apply:**

- **the shoulder width is less than 5 feet.**
- **the median width is 14 feet or less.** See Index 305.5 for further paved median guidance.
- **on roads with less than two lanes in the direction of travel and there is a sustained (greater than 1 mile in length) grade of over 4 percent without a truck climbing lane.**
- **the shoulders are adjacent to exclusive truck or bus only lanes, or weigh station ramps.** This standard does not apply to mixed use (automobile plus bus) lanes, including high-occupancy vehicle (HOV) and toll (HOT) lanes.

The shoulder may also be engineered to match the TI of the adjacent traffic lane provided that:

- There is an identified plan (such as Regional Transportation Plan, Metropolitan Transportation Plan, Interregional Improvement Plan) to convert a shoulder into a traffic lane within the next 20 years.
- The shoulder is designed following the lane width and cross slope guidance in Topic 301.
- Agreement is obtained by the Program Fund Manager or Agency funding the project.

When the above conditions apply and the shoulder and lane will both be constructed as part of the same project, the shoulder pavement structure should match the adjacent traffic lane for ease of construction. For asphalt pavements, the thickness of the shoulder surface course may be tapered from the lane surface course thickness to the shoulder pavement edge thickness of no less the 0.35 foot to address different cross slope conditions (see Figure 613.5A).

For all other cases, the minimum TI for the shoulder shall match the TI of the adjacent traffic lane for the first 2 feet of the outside shoulder width and 1 foot of the inside shoulder measured from the edge of traveled way. See Figure 613.5B.

For the remaining width of the shoulder, the TI shall:

- **be no less than 2 percent of the projected ESALs of the adjacent traffic lane or a TI of 5, whichever is greater.**
- **not exceed 9.**

Treated permeable bases needed to perpetuate an existing treated permeable base under the adjacent lane may be included underneath the pavement. Non-permeable treated bases, such as lean concrete base, are not to be included underneath the pavement.

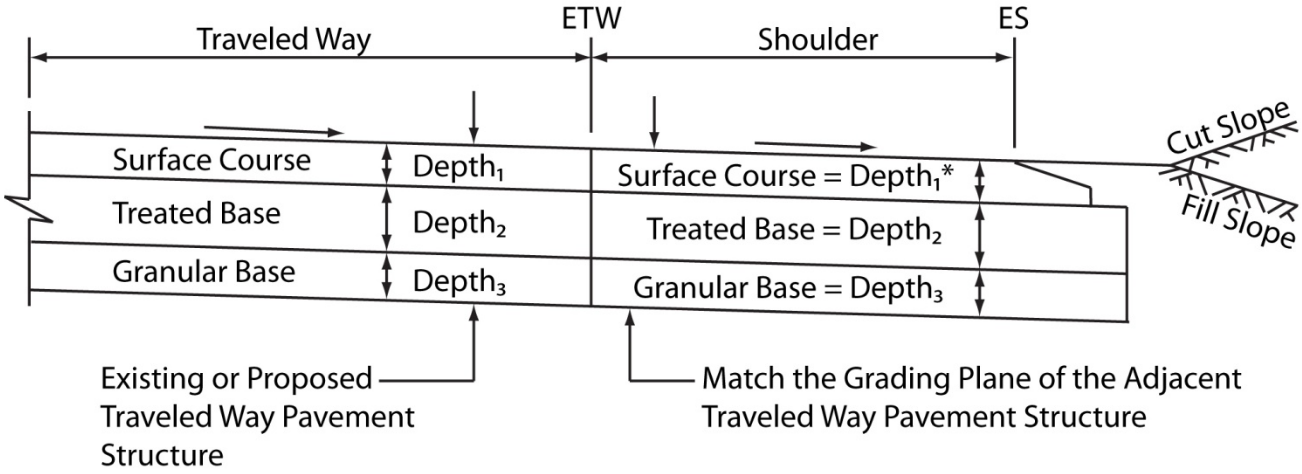
The total depth of the shoulder pavement structure (depth from the surface to the subgrade) shall match the pavement structure grading plane of the adjacent traffic lane.

Matching the grading plane of the shoulder pavement structure to that of the adjacent traffic lane can be accomplished by increasing the depth of the aggregate base and/or subbase as needed (see Figure 613.5B). This will provide a path for water in the pavement structure to drain away from the lane and into the shoulder. It can also provide a more cost effective means to upgrade the shoulder to a traffic lane in the future. Although using a thinner overall shoulder pavement structure than the traveled way requires less material and may appear to reduce construction costs, the added costs of time and labor to the Contractor to build the step between the

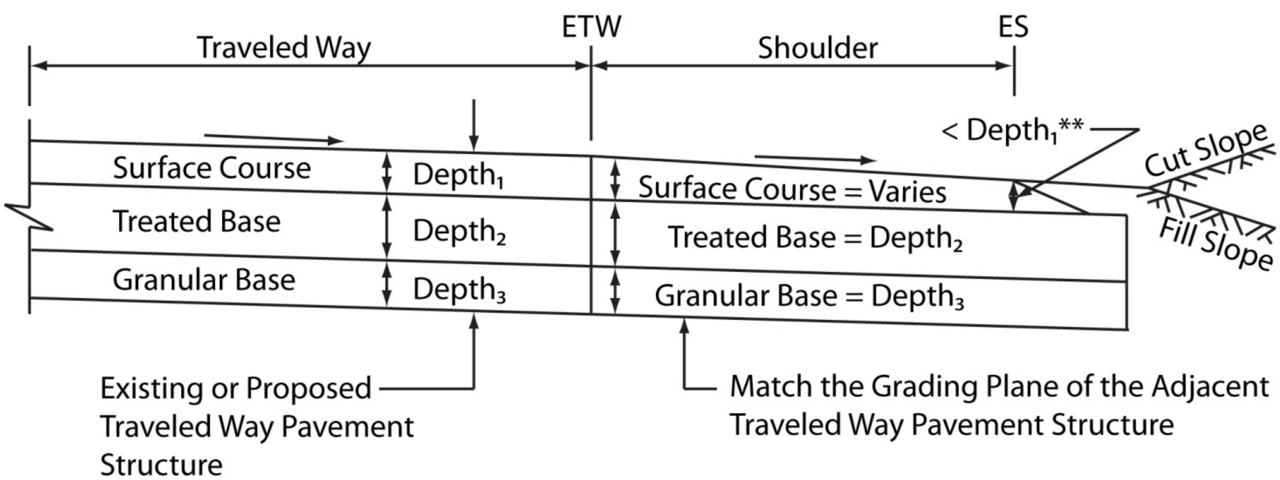
Figure 613.5A

Shoulder Design for TI Equal to Adjacent Lane TI

Shoulder Pavement Structure is the Same as Traveled Way Structure



Variable Shoulder Thickness Option for Asphalt Pavement



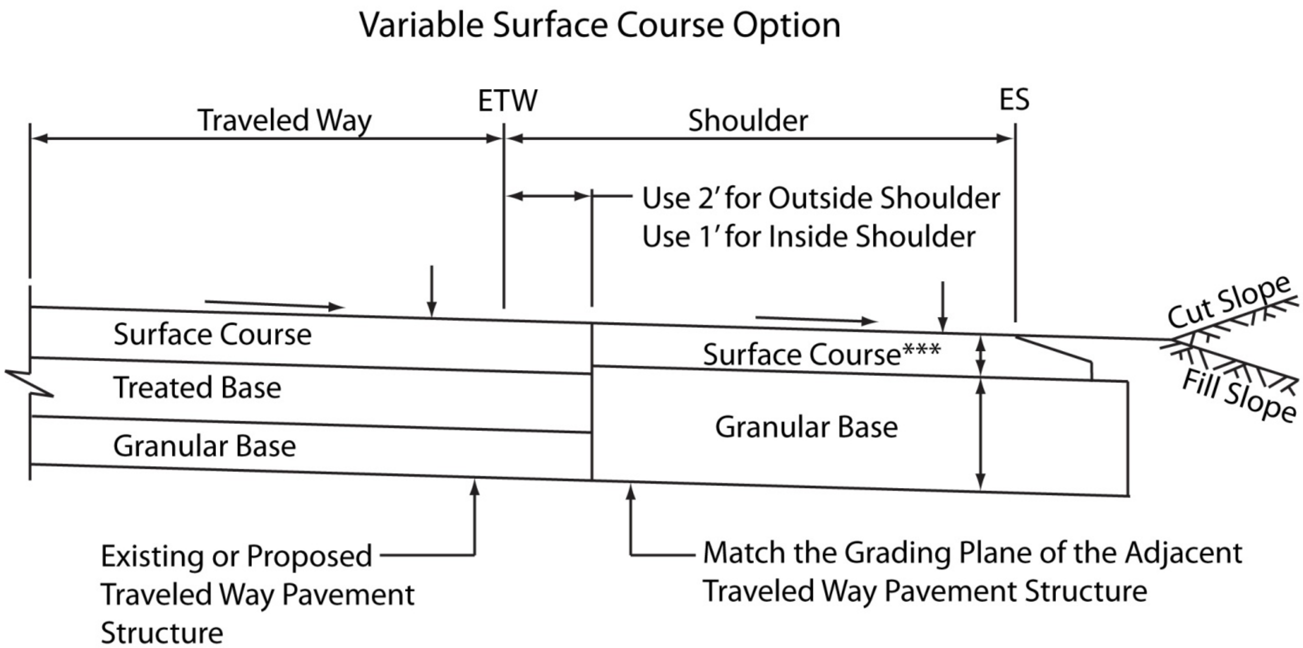
NOTES:

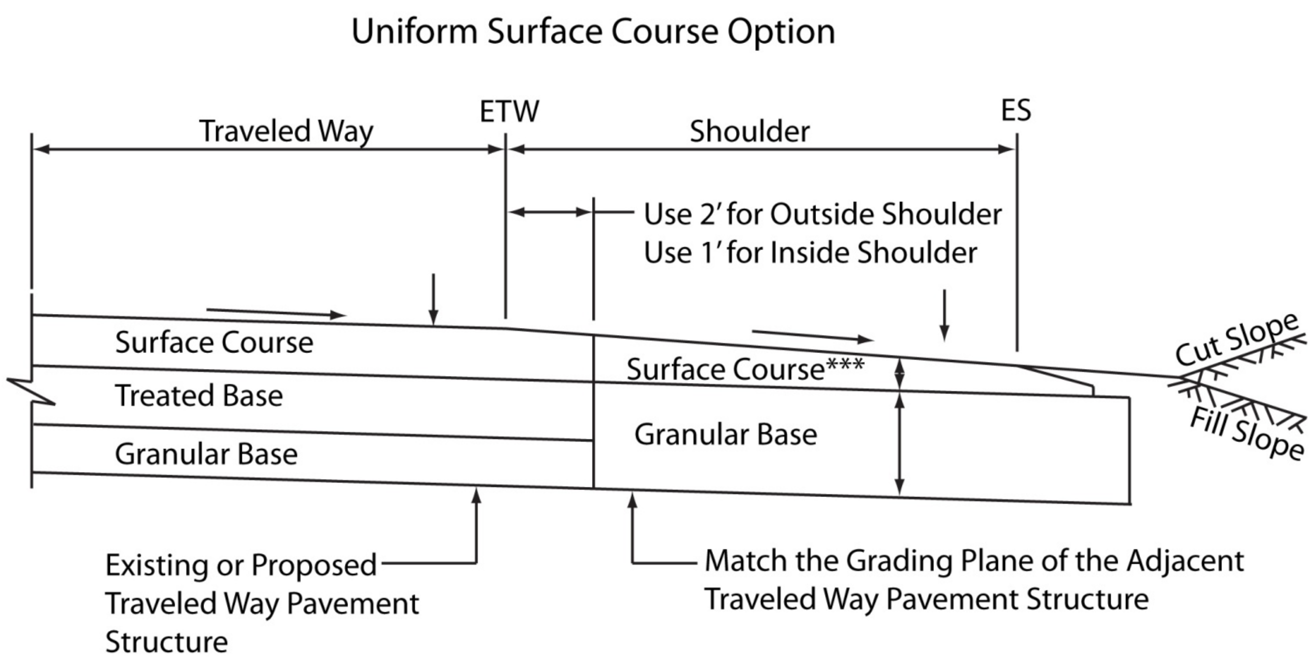
* Applies to concrete and asphalt pavements.

** For asphalt pavement, minimum thickness of surface course ≥ 0.35’.

Figure 613.5B

Shoulder Design for TI Less Than Adjacent Lane TI





NOTES:

*** For rigid pavement, minimum thickness of surface course is ≥ 0.60’ (0.75’ for High Mountain or High Dessert Climate Region)

For flexible pavement, minimum thickness of surface course is ≥ 0.35’ traveled way and shoulder can offset any perceived savings from reduced materials.

For asphalt shoulders, the thickness of the asphalt layer (not including nonstructural wearing surface) should not be less than 0.35 foot or the thickness of the asphalt layer of the adjacent traffic lane, whichever is less.

For concrete shoulders, a pavement structure of 0.70 foot undoweled jointed plain concrete pavement (0.85 foot in High Mountain and High Desert climate regions) over aggregate base is sufficient to meet the requirement of the TI not exceeding 9.0 and provide adequate structure for maintenance equipment and temporary traffic detours.

An alternate shoulder design is to taper the surface course from the surface course thickness of the adjacent traffic lane to no less than 0.60 foot (0.75 foot in High Mountain and High Desert climate regions) for concrete and 0.35 foot for asphalt at the edge of shoulder (see Figure 613.5B).

Bases and subbases for new or reconstructed shoulders should extend at least 1 foot beyond the edge of shoulder as shown in Figures 613.5A and 613.5B.

(c) Widening.

Existing shoulders do not need to be replaced or upgraded to new construction or reconstruction standards as part of a shoulder widening project unless the following conditions exist:

- Adding or widening lanes will require removal of all or a portion of the existing shoulder.
- The existing shoulder of 5 feet or less in width is being widened and the existing shoulder does not meet the current standards for new construction or reconstruction. For shoulders wider than 5 feet, the District and Program Fund Manager/Agency determines whether to reconstruct the entire shoulder to new construction or reconstruction standards, or match the pavement structure of the existing shoulder.
- There is an identified plan that the widened shoulder will be converted or replaced with a traffic lane within 20 years.
- The widened shoulder will be used as a temporary detour as discussed in Index 613.5(2)(f).

For all other cases, widening of the existing shoulder should match the pavement structure of the existing shoulder. For shoulders left in place, repair any existing distresses prior to overlaying.

(d) Pavement Preservation.

Shoulder preservation should be done in conjunction with work on the adjacent traffic lanes to assure that the shoulder pavement structure will meet the performance requirements stated in Index 613.5(2)(a). Shoulders can be preserved by:

- Sealing cracks greater than ¼ inch in width,
- Grinding out rolled up sections next to concrete pavement,
- Fog or slurry sealing asphalt surfaces,
- Limiting digouts of failed locations.

For CAPM projects, the following additional strategies can be considered if warranted:

- Milling and replacing 0.15 foot of oxidized and cracked surfaces can be considered either prior to an overlay or as a stand-alone action.
- Grinding of concrete shoulders if the adjacent traffic lane is being ground.

Shoulder preservation strategies should be identified and discussed with District Maintenance and the Headquarters Pavement Reviewer during the scoping phase of the project or whenever a change in strategy is proposed.

(e) Roadway Rehabilitation.

The goal in roadway rehabilitation projects is to maintain existing shoulders wherever possible. The TI is not a consideration in choosing the shoulder rehabilitation strategy unless it has been determined that the shoulder needs to be replaced for one of the following reasons:

- The shoulder will be used to temporarily detour traffic during construction and the existing shoulder does not provide adequate structure to handle the expected loads.
- The adjacent lane is being replaced as part of the project. In this situation, if the shoulder is wider than 5 feet, replace only two feet of the outside shoulder (1 foot of inside shoulder) adjacent to the traffic lane. For shoulders 5 feet wide or less, replace the entire shoulder.
- The existing shoulder exhibits extensive distress and/or settlement and it is agreed to by the Headquarters Pavement Reviewer that replacement is the only viable option.

For replacements other than temporary traffic detours, use the standards for new construction and reconstruction in Index 613.5(2)(b). For temporary traffic detours, see Index 613.5(2)(f) for further discussion.

Regardless of whether or not the TI is considered, shoulder rehabilitation repairs of the existing shoulder are often necessary and should be done in conjunction with work on the adjacent traffic lanes to assure that the shoulder pavement will meet the performance requirements stated in Index 613.5(2)(a).

Existing asphalt shoulders can typically be maintained as part of a rehabilitation project by milling and replacing 0.15 feet of asphalt surface plus digouts of failed areas to remove oxidized layers. This can be done either prior to an overlay or to maintain the existing surface. Where the existing shoulders have little to no cracking and are older than 3 years from the last treatment, a fog seal or slurry seal with digouts is all that is needed.

Existing concrete shoulders typically only require sealing any unsealed cracks ½ inch or wider or replacing the joint seals. Shoulders should be sealed if the adjacent traffic lanes are sealed. If shoulders are spalled, the spalls should be repaired and any shattered slabs replaced. Grinding should not be done, even if the shoulder is faulted or curled unless the adjacent traffic lane is also being ground.

Shoulder rehabilitation strategies should be identified and discussed with District Maintenance and the Headquarters Pavement Reviewer during the scoping phase of the project or whenever a change in strategy is proposed.

(f) Temporary Detours.

When existing shoulders will be used to stage traffic during construction, the existing shoulder pavement structure should be checked for structural adequacy. If the existing shoulder is not structurally adequate or if it is a new shoulder, calculate the TI based on the actual truck traffic expected to be encountered during construction. Design the shoulder based on the requirements for new or reconstructed shoulders in Index 613.5(2)(b) except the TI may exceed 9. Do not use treated bases for temporary detours. For existing shoulders, remove the surface course layer and replace with a new surface course sufficiently thick enough to support temporary traffic loads.

(g) Conversion to Lane.

If a decision has been made to convert an existing shoulder to a portion of a traffic lane, a deflection study must be performed to determine the structural adequacy of the in place asphalt shoulder. The condition of the existing shoulder must also be evaluated for undulating grade, rolled-up hot mix asphalt at the rigid pavement joint, surface cracking, raveling, brittleness, oxidation, etc.

The converted facility must provide a roadway that is structurally adequate for the proposed pavement design life. This is necessary to eliminate or minimize the likelihood of excessive maintenance or rehabilitation being required in a relatively short time because of inadequate structural strength and deterioration of the existing pavement structure.

If the existing shoulder is determined to be structurally inadequate for the proposed pavement design life, then the shoulder should be upgraded or replaced in accordance with the standards for new construction and reconstruction discussed in Index 613.5(2)(b).

(h) Other.

- Tracking and Sweep Width Lines.

  For projects where the tracking width and sweep width lines are shown to encroach onto the paved shoulders, the shoulder pavement structure must be engineered to sustain the weight of the design vehicle. 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 Topic 404 for design vehicle guidance.

- Minimizing Worker Exposure.

  Consult with District Maintenance and the Headquarters Program Advisor during the scoping phase on options for minimizing maintenance worker exposure to maintain shoulders.

- Concrete shoulders and asphalt pavement structure.

  Do not place concrete shoulders adjacent to asphalt pavement structure.

(3) *Intersections.* Future AADTT and TI's for intersections should be determined for each approach the same way as for mainline traffic. At some intersections, the level of truck/transit traffic from all approaches may add more loads on the pavement than what the mainline pavement was designed for. Separate ESAL/TI or load spectra calculations should be performed at intersections when any of the following criteria apply:

- Two or more State highways intersect (including ramps to/from State highways)
- Truck traffic on the local road exceeds 25 percent of the truck traffic on the State highway.
- Ramp connecting a State highway to a local road is classified as Medium or Heavy as described in Index 613.5(1)(c).

In these cases, combine the traffic counts/ESALs of the approaches to calculate the TI or load spectra for all approaches combined. If the resulting TI or load spectra are higher than what is calculated for the mainline, then the intersections will need to be engineered using the combined TI or load spectra.

For all roundabout designs, look at the traffic projections for each turning movement of each leg of the roundabout, then, sum up the truck/transit traffic volumes using each quadrant of the roundabout. From the total truck traffic volume, generate an ESAL/TI or load spectra for each quadrant. Choose the quadrant with the highest TI or load spectra to design the entire roundabout.

Special attention should be given to truck and transit traffic behavior (turning and stopping) to determine the loading patterns and to select the most appropriate materials.

The limits for engineering pavement at an intersection should include intersection approaches and departures, to the greater of the following distances:

- For signalized intersections, the limits of the approach should extend past the furthest set of signal loop detectors where trucks do the majority of their braking; or
- For "STOP" controlled intersections the limits for the approach should be long

enough to cover the distance trucks will be braking and stopping either at the stop bar or behind other trucks and vehicles; or

- 100 feet.

The limits for the intersection departures should match the limits of the approach in the opposing lane to address rutting caused by truck acceleration.

For further assistance on this subject, contact either your District Materials Engineer, or Headquarters Pavement Program – Office of Concrete Pavement and Pavement Foundations.

(4) *Roadside Facilities*. The pavement for safety roadside rest areas, including parking lots, should meet or exceed the TI requirements found in Table 613.5B for a 20-year pavement design life for new/reconstructed or rehabilitated pavements.

Table 613.5B
Minimum TI's for Safety Roadside Rest Areas

Facility Usage	Minimum TI (20-Year)
Truck Ramps & Roads	8.0 (1)
Truck Parking Areas	6.0 (1)
Auto Roads	5.5
Auto Parking Areas	5.0

NOTE:

(1) For safety roadside rest areas next to all Interstates and those State Routes with AADTT greater than 15,000 use Table 613.5A medium truck traffic for truck ramps, truck roads, and a minimum TI of 9.0 for truck parking areas.

Topic 614 - Soil Characteristics

614.1 Engineering Considerations

California is a geologically active state with a wide variety of soil types throughout. Thorough understanding of the native soils in a project area is essential to properly engineer or update a highway facility.

Subgrade is the natural soil or rock material underlying the pavement structure. Unlike concrete and steel whose characteristics are fairly uniform, the engineering properties of subgrade soils may vary widely over the length of a project.

Pavements are engineered to distribute stresses imposed by traffic to the subgrade. For this reason, subgrade condition is a principal factor in selecting the pavement structure. Before a pavement is engineered, the structural quality of the subgrade soils must be evaluated to ensure that it has adequate strength to carry the predicted traffic loads during the design life of the pavement. The pavement must also be engineered to limit the expansion and loss of density of the subgrade soil.

614.2 Unified Soil Classification System (USCS)

The USCS classifies soils according to their grain size distribution and plasticity. Therefore, only a sieve analysis and Atterberg limits (liquid limit, plastic limit, and plasticity index) are necessary to classify a soil in this system. Based on grain size distribution, soils are classified as either (1) coarse grained (more than 50 percent retained on the No. 200 sieve), or (2) fine grained (50 percent or more passes the No. 200 sieve). Coarse grained soils are further classified as gravels (50 percent or more of coarse fraction retained on the No. 4 sieve) or sands (50 percent or more of coarse fraction passes the No. 4 sieve); while fine grained soils are classified as inorganic or organic silts and clays and by their liquid limit (equal to or less than 50 percent, or greater than 50 percent). The USCS also includes peat and other highly organic soils, which are compressible and not recommended for roadway construction. Peat and other highly organic soils should be removed wherever possible prior to placing the pavement structure.

The USCS based on ASTM D 2487 is summarized in Table 614.2.

614.3 California R-Value

The California R-value is the measure of resistance to deformation of the soils under wheel loading and saturated soil conditions. It is used to determine the bearing value of the subgrade. Determination of R-value for subgrade is provided under California Test (CT) 301. Typical R-values used by the

Table 614.2
Unified Soil Classification System (from ASTM D 2487)

Major Classification Group	Sub-Groups		Classification Symbol	Description
Coarse Grained Soils More than 50% retained on the No. 200 sieve	**Gravels** 50% or more of coarse fraction retained on the No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of coarse fraction passes the No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine Grained Soils More than 50% passes the No. 200 sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

Prefix: G = Gravel, S = Sand, M = Silt, C = Clay, O = Organic
Suffix: W = Well Graded, P = Poorly Graded, M = Silty, L = Clay, LL < 50%, H = Clay, LL > 50%

Department range from five for very soft material to 80 for treated base material.

The California R-value is determined based on the following separate measurements under CT 301:

- The exudation pressure test determines the thickness of cover or pavement structure required to prevent plastic deformation of the soil under imposed wheel loads.
- The expansion pressure test determines the pavement thickness or weight of cover required to withstand the expansion pressure of the soil.

Because some soils, such as coarse grained gravels and sands, may exhibit a higher California R-value test result than would normally be required for pavement design, the California R-value for subgrade soils used for pavement design should be limited to no more than 50 unless agreed to otherwise by the District Materials Engineer. Local experience with these soils should govern in assigning R-value on subgrade.The California R-value of subgrade within a project may vary substantially but cost and constructability should be considered in specifying one or several California R-value(s) for the project. Engineering judgment should be exercised in selecting appropriate California R-values for the project to assure a reasonably "balanced design" which will avoid excessive costs resulting from over conservatism. The following should be considered when selecting California R-values for a project:

- If the measured California R-values are in a narrow range with some scattered higher values, the lowest California R-value should be selected for the pavement design.
- If there are a few exceptionally low California R-values and they represent a relatively small volume of subgrade or they are concentrated in a small area, it may be more cost effective to remove or treat these materials.
- Where changing geological formations and soil types are encountered along the length of a project, it may be cost-effective to design more than one pavement structure to accommodate major differences in R-values that extend over a considerable length. Care should be exercised to avoid many variations in the pavement structure that may result in increased construction costs that exceed potential materials cost savings.

614.4 Expansive soils

With an expansive subgrade (Plasticity Index greater than 12), special engineering or construction considerations will be required. Engineering alternatives, which have been used to compensate for expansive soils, are:

(a) Treating expansive soil with lime or other additives to reduce expansion in the presence of moisture. Lime is often used with highly plastic, fine-grained soils. When mixed and compacted, the plasticity and swelling potential of clay soils are reduced and workability increased, as lime combines with the clay particles. It also increases the California R-value of the subgrade. Soil treated with lime is considered to be lime treated subbase. Lime treated subbase is discussed further in Chapter 660.

(b) Replacing the expansive material with a non-expansive material to a depth where the seasonal moisture content will remain nearly constant.

(c) Providing a pavement structure of sufficient thickness to counteract the expansion pressure.

(d) Utilizing two-stage construction by placing a base or subbase to permit the underlying material to expand and stabilize before placing leveling and surface courses.

(e) Stabilizing the moisture content by minimizing the access of water through surface and subsurface drainage and the use of a waterproof membrane (i.e., geomembrane, asphalt saturated fabric, or rubberized asphalt membrane).

(f) Relocating the project alignment to a more suitable soil condition.

Treatment (e) is considered to be the most effective approach if relocation is not feasible such as in the San Joaquin Delta. The District Materials Engineer determines which treatment(s) is/are practical.

The California R-value of the subgrade can be raised above 10 by treatment to a minimum depth of 0.65 foot with an approved stabilizing agent such as lime, cement, asphalt, or fly ash. Native soil samples should be taken, treated, and tested to determine the California R-value for the treated subgrade. For pavement structure design, the maximum California R-value that can be specified for treated subgrade regardless of test results is 40. Treating the subgrade does not eliminate or reduce the required aggregate subbase for rigid or composite pavements in the rigid pavement catalog (see Topic 623). With HMA, treated subgrade can be substituted for all or part of the required aggregate subbase layer. Since aggregate subbase has a gravel factor (G_f) of 1.0, the actual thickness and the gravel equivalent (GE) are equal. When the treated subgrade is substituted for aggregate subbase for flexible pavements, the actual thickness of the treated subgrade layer is obtained by dividing the GE by the appropriate G_f. The G_f is determined based on unconfined compressive strength (UCS) of the treated material as follows:

$$G_f = 0.9 + \frac{UCS(psi)}{1000}$$

This equation is only valid for UCS of 300 psi or more. The gravel factor G_f should be a minimum of 1.2. The maximum G_f allowed using this equation is 1.7. Because the treatment of subgrade soil may be less expensive than the base material, the calculated base thickness can be reduced and the treated subgrade thickness increased because of cost considerations. The base thickness is reduced by the corresponding gravel equivalency provided by the lime treated subgrade soil or subbase. The maximum thickness of lime treated subgrade is limited to 2 feet.

Rigid or composite pavement should not be specified in areas with expansive soils unless the pavement has been adequately treated to address soil expansion. Flexible pavement may be specified in areas where expansive soils are present with the understanding that periodic maintenance would be required.

The District Materials Engineer should be contacted to assist with the selection of the most appropriate method to treat expansive soils for individual projects. Final decision as to which treatment to use rests with the District.

614.5 Subgrade Enhancement Geotextile (SEG)

The placement of subgrade enhancement geotextile (SEG), formerly called subgrade enhancement fabric (SEF), below the pavement will provide subgrade enhancement by bridging soft areas and providing a separation between soft subgrade fines susceptible to pumping and high quality subbase or base materials. On weak subgrades, the use of SEG can provide for stabilization (the coincident function of separation and reinforcement). As the soft soil undergoes deformation, properly placed geotextile when stretched will develop tensile stress. Locations that may require placement of SEG include areas with the following soil characteristics:

- Poor (low strength) soils which are classified in the unified soil classification system (USCS) as sandy clay (SC), silty clay (CL), high plastic clay (CH), silt (ML), high plasticity or micaceous silt (MH), organic silt (OL), organic clay (OH), and peat & mulch (PT).
- Low undrained shear strength (equivalent to California R-value <20).
- High water table, and high soil sensitivity.

Subgrade soils with R-value <20 are considered poor or weak soils and require SEG to provide reinforcement as the primary function and separation as the secondary function. However, pavements constructed over subgrade soils with R-value up to 40 can especially benefit from separation if the soil contains an appreciable amount of fines, depending on type and treatment of the base layer. The SEG when placed with aggregate subbase provides a working platform for access of construction equipment, mainly on subgrades with R-values of 5 to 10.

The use of SEG on weak subgrades (with R-value <20) can raise the effective R-value of such soils to 20. Therefore, the benefit of using SEG on such weak soils can be realized though using thinner aggregate bases or subbases in flexible pavement design. Likewise, SEG can also affect the design of rigid pavements by providing a stronger subgrade system.

The method of determining the functions realized from the use of SEG and the selection of the appropriate properties of the SEG based on project specifics are explained in the "Subgrade Enhancement Geotextile Guide" on the Department Pavement website.

614.6 Other Considerations

(1) *Fill.* Because the quality of excavated material may vary substantially along the project length, the pavement design over a fill section should be based on the minimum California R-value or unified soil classification of the material that is to be excavated as part of the project. If there is any excavated material that should not be used, it should be identified in the Materials Report and noted as appropriate in the PS&E.

(2) *Imported Borrow.* Imported borrow is used in the construction of embankments when sufficient quantity of quality material is not available. The pavement design should be based on the minimum California R-value of imported borrow or excavated fill material on the project. When imported borrow of desired quality is not economically available or when all of the earthwork consists of borrow, the California R-value specified for the borrow becomes the design R-value. Since no minimum California R-value is required by the Standard Specifications for imported borrow, a minimum R-value for the imported borrow material placed within 4 feet of the grading plane must be specified in the Materials Report and in the project plans.

(3) *Compaction.* Compaction is densification of the soil by mechanical means. The Standard Specifications require no less than 95 percent relative compaction be obtained for a minimum depth of 2.5 feet below finished grade for the width of the traveled way and auxiliary lanes plus 3 feet on each side. The 2.5 feet depth of compaction should not be waived for the traveled way, auxiliary lanes, and ramps on State highways.

These specifications sometimes can be waived by special provision with approval from the District Materials Engineer, when any of the following conditions apply:

- A portion of a local road is being replaced with a stronger pavement structure.
- Partial-depth reconstruction is specified.
- Existing buried utilities would have to be moved.
- Interim widening projects are required on low-volume roads, intersection channelization, or frontage roads.

Locations where the 2.5 feet of compaction depth is waived must be shown on the typical cross sections of the project plan. If soft material below this depth is encountered, it must be removed and replaced with suitable excavated material, imported borrow or subgrade enhancement fabric. Location(s) where the Special Provisions apply should be shown on the typical cross section(s).

Topic 615 - Climate

The effects that climate will have on pavement must be considered as part of pavement engineering. Temperatures will cause pavements to expand and contract creating pressures that can cause pavements to buckle or crack. Binders in flexible pavements will also become softer at higher temperatures and more brittle at colder temperatures. Precipitation can increase the potential for water to infiltrate the base and subbase layers, thereby resulting in increased susceptibility to erosion and weakening of the pavement structural strength. In freeze/thaw environments, the expansion and contraction of water as it goes through freeze and thaw cycles, plus the use of salts, sands, chains, and snow plows, create additional stresses on pavements. Solar radiation can also cause some pavements to oxidize. To help account for the effects of various climatic conditions on pavement performance, the State has been divided into the following nine climate regions.

- North Coast
- Central Coast
- South Coast
- Low Mountain
- High Mountain
- South Mountain
- Inland Valley

- Desert
- High Desert

Figure 615.1 provides a representation of where these regions are. A more detailed map along with a detailed list of where State routes fall within each climate region can be found on the Department Pavement website.

In conjunction with this map, designs, standards, plans, and specifications have been and are being developed to tailor pavement standards and practices to meet each of these climatic conditions. The standards and practices found in this manual, the Standard Plans, Standard Specifications, and Special Provisions should be considered as the minimum requirements to meet the needs of each climate region. Districts may also have additional requirements based on their local conditions. Final decision for the need for any requirements that exceed the requirements found in this manual, the Standard Plans, Standard Specifications, and Standard Special Provisions rests with the District.

Topic 616 - Existing Pavement Type and Condition

The type and condition of pavement on existing adjacent lanes or facilities should be considered when selecting new pavement structures or rehabilitation/preservation strategies. The selection process and choice made by the engineer is influenced by their experience and knowledge of existing facilities in the immediate area that have given adequate service. Providing continuity of existing pavement type will also ensure consistency in maintenance operations.

In reviewing existing pavement type and condition, the following factors should be considered:

- Type of pavement on existing adjacent lanes or facilities
- Performance of similar pavements in the project area
- Corridor continuity
- Maintaining or changing grade profile
- Existing pavement widening with a similar material
- Existing appurtenant features (median barriers, drainage facilities, curbs and dikes, lateral and overhead clearances, and structures which may limit the new or rehabilitated pavement structure.)

Topic 617 - Materials

617.1 Availability of Materials

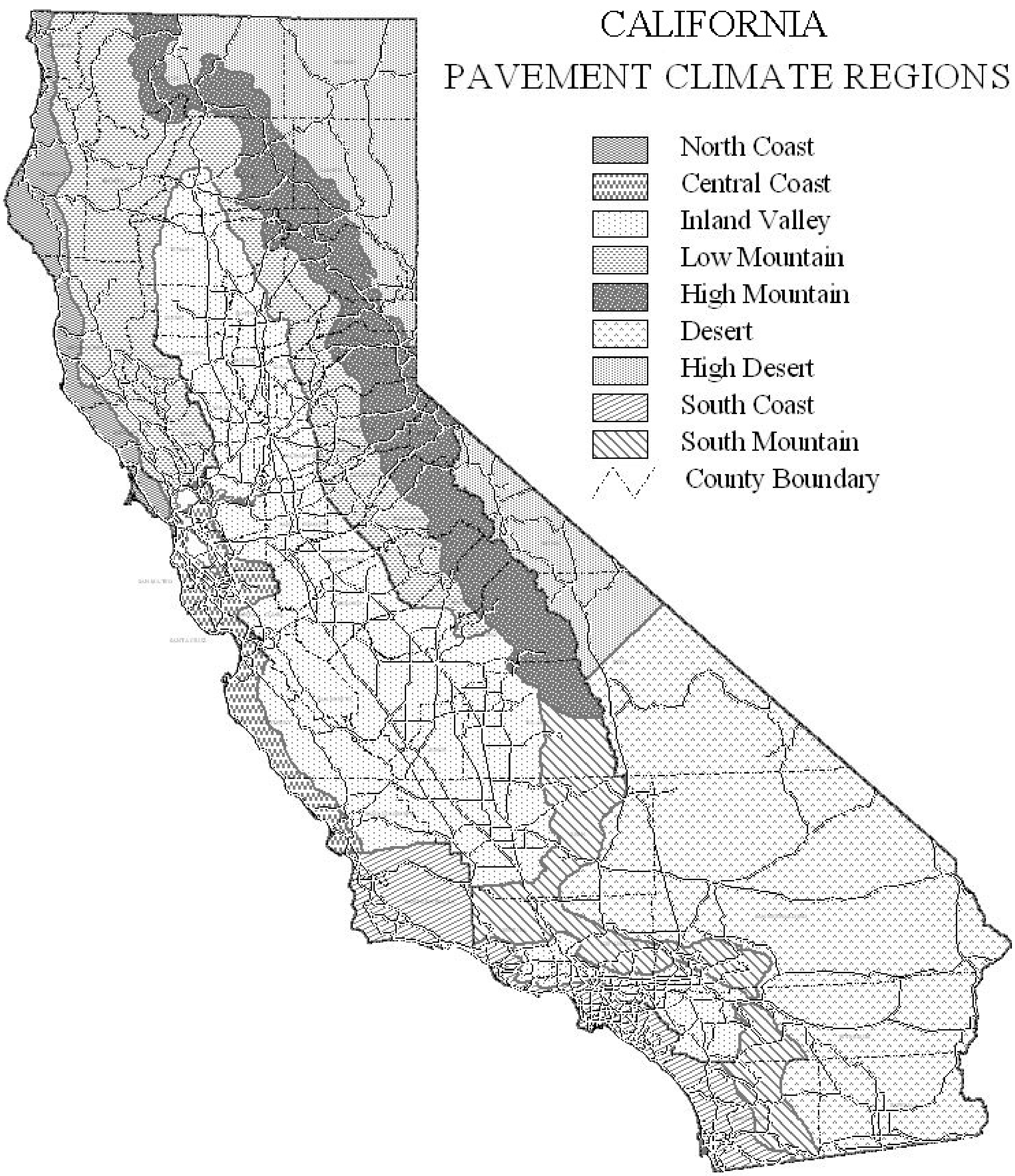
The availability of suitable materials such as subbase and base materials, aggregates, binders, and cements for pavements should be considered in the selection of pavement type. The availability of commercially produced mixes and the equipment capabilities of area contractors may also influence the selection of pavement type, particularly on small widening, reconstruction or rehabilitation projects. Materials which are locally available or require less energy to produce and transport to the project site should be used whenever possible.

617.2 Recycling

The Department encourages and seeks opportunities to utilize recycled materials in construction projects whenever such materials meet the minimum engineering standards and are economically viable. Accordingly, consideration should be given on every project to use materials recycled from existing pavements as well as other recycled materials such as scrap tires. Existing pavements can be recycled for use as subbase and base materials, or as a partial substitute for aggregate in flexible surface course for rehabilitation or reconstruction projects. The decision to use recycled materials however should be made on a case-by-case basis based on a thorough evaluation of material properties, performance experience in prior projects, benefit/cost analysis, and engineering judgment. Additional information on use of recycled pavements is available in Index 110.11 and on the Department Pavement website.

Candidates for recycling flexible pavement surface courses are those with uniform asphalt content. The existence of heavy crack-sealant, numerous patches, open-graded friction course, and heavy seal coats make the new recycled hot mix asphalt design inconsistent thereby resulting in mix properties that are more difficult to control. To avoid this problem when it occurs and still use the

Figure 615.1
Pavement Climate Regions



NOTE: Map is shown for reference only.
See the Department Pavement website for the detailed map to use.

recycle option, for flexible pavement, a minimum of 0.08 foot should be milled off prior to the recycling operation. Light crack sealing (less than 5 percent of the pavement) or a uniform single seal coat will not influence the pavement engineering sufficiently to require milling.

The Department has established a minimum mill depth of 0.15 foot for recycling flexible pavement surface courses. Since existing surface course thickness will have slight variations, the recycling strategy should leave at least the bottom 0.15 foot of the existing flexible surface course in place. This is to insure the milling machine does not loosen base material and possibly contaminate the recycled material. As mentioned in Index 110.11(2), recycling of existing hot mix asphalt must be considered, in all cases, as an alternative to placing 100 percent new hot mix asphalt.

Topic 618 - Maintainability and Constructibility

618.1 Maintainability

Maintainability is the ability of a highway facility to be restored in a timely and cost-effective way with minimal traffic exposure to the workers and minimal traffic delays to the traveling public. It is an important factor in the selection of pavement type and pertinent appurtenances. Maintainability issues should be considered throughout the project development process to ensure that maintenance needs are adequately addressed in the engineering and construction of the pavement structure. For example, while a project may be constructible and built in a timely and cost-effective manner, it may create conditions requiring increased worker exposure and increased maintenance effort that is more expensive and labor intensive to maintain. Another example is the pavement drainage systems that need frequent replacement and often do not provide access for cleanout.

Besides the minimum considerations for the safety of the public and construction workers found in this manual, the Standard Specifications, and other Department manuals and guidance, greater emphasis should also be placed on the safety of maintenance personnel and long-term maintenance costs over the service life for the proposed project rather than on constructibility or initial costs. Minimizing exposure to traffic through appropriate pavement type selection and sound engineering practices should always be a high priority. The District Maintenance Engineer and Maintenance Supervisor responsible for maintaining the project after it is built should be consulted for recommendations on addressing maintainability.

618.2 Constructibility

Construction issues that influence pavement type selection include: size and complexity of the project, stage construction, lane closure requirements, traffic control and safety during construction, construction windows when the project must be completed, and other constructibility issues that have the potential of generating contract change orders.

The Project Engineer must be cognizant of the issues involved in constructing a pavement, and provide plans and specifications that both meets performance standards and requirements. The Construction Engineer for the area where the pavement will be built should be consulted regarding constructibility during the project development process. The recommendations given by Construction should be weighed against other recommendations and requirements for the pavement. Constructibility recommendations should be accommodated where practical, provide minimum performance requirements, safety, and maintainability. Some constructibility items that should be addressed in the project include:

- Clearance width of paving machines to barriers and hinge points.
- Access for delivery trucks and construction equipment.
- Public safety and convenience.
- Time and cost of placing multiple thin lifts of different materials as opposed to thicker lifts of a single material. (For example, sometimes it is more efficient and less costly to place one thick lift of aggregate base rather than two thin lifts of aggregate base and subbase).
- The impact of combined lifts of different materials on long-term performance or maintenance of the pavement. (For example, it may seem to be a good idea to combine layers of portland cement concrete and lean concrete

base into a single layer to make it easier to construct, but combining these layers has a negative impact on the pavement performance and will lead to untimely failure).

- Time and cost of using multiple types of hot mix asphalt on a project in an area away from commercial hot mix asphalt sources.

Topic 619 - Life-Cycle Cost Analysis

619.1 Life-Cycle Cost Analysis

Life-cycle cost analysis (LCCA) is a useful tool for comparing the value of alternative pavement structures and strategies. LCCA is an economic analysis that compares initial cost, future cost, and user delay cost of different pavement alternatives. LCCA is an integral part of the decision making process for selecting pavement type and design strategy. It can be used to compare life-cycle cost for:

- Different pavement types (rigid, flexible, composite).
- Different rehabilitation strategies.
- Different pavement design lives (20 vs. 40, etc).

LCCA comparisons must be made between properly engineered, viable pavement structures that would be approved for construction if selected. The alternatives being evaluated should also have identical improvements. For example, comparing 20-year rehabilitation vs. 40-year rehabilitation or flexible pavement new construction vs. rigid pavement new construction, provide an identical improvement. Conversely, comparing pavement rehabilitation to new construction, or pavement overlay to pavement widening are not identical improvements.

LCCA can also be useful to determine the value of combining several projects into a single project. For example, combining a pavement rehabilitation project with a pavement widening project may reduce overall user delay and construction cost. In such case, LCCA can help determine if combining projects can reduce overall user delay and construction cost for more efficient and cost-effective projects. LCCA could also be used to identify and measure the impacts of splitting a project into two or more projects.

LCCA must conform to the procedures and data in the Life-Cycle Cost Analysis Procedures Manual. LCCA must be completed for any project with a pavement cost component except for the following:

- Major maintenance projects.
- Minor A and Minor B projects.
- Projects using Permit Engineering Evaluation Reports (PEER).
- Maintenance pullouts.
- Landscape.

For the above exempted projects, the Project Manager and the Project Development Team (PDT) will determine on a case-by-case basis if and how a life-cycle cost analysis should be performed and documented. Information on how to document life-cycle costs can be found in the Department's Project Development Procedures Manual, Chapter 8.

CHAPTER 620
RIGID PAVEMENT

Topic 621 - Types of Rigid Pavements

Index 621.1 Jointed Plain Concrete Pavement (JPCP)

JPCP is the most common type of rigid pavement used by the Department. JPCP is engineered with longitudinal and transverse joints to control where cracking occurs in the slabs (see Figure 621.1). JPCPs do not contain steel reinforcement, other than tie bars and dowel bars (see Index 622.4 for tie bars and dowel bars). Additional guidance for JPCP can be found in the "Guide for Design and Construction of New Jointed Plain Concrete Pavements" on the Department Pavement website.

621.2 Continuously Reinforced Concrete Pavement (CRCP)

Although the Department has used CRCP on a limited basis in the past, CRCP is still a relatively new concept to California. For this reason, the Department has decided not to use CRCP for TIs less than 11.5 or in High Mountain and High Desert climate regions. Since CRCP uses reinforcing steel rather than weakened plane joints for crack control, saw cutting of transverse joints is not required for CRCP. Longitudinal joints are still used. Transverse random cracks are expected in the slab, usually at 3-foot to 5-foot intervals (see Figure 621.1). The continuous reinforcement in the pavement holds the cracks tightly together. CRCP typically costs more initially than JPCP due to the added cost of the reinforcement. However, CRCP is typically more cost-effective over the life of the pavement on high volume routes due to improved long-term performance and reduced maintenance. Because there are no sawn transverse joints, properly built CRCP should have better ride quality and less maintenance than JPCP. Additional CRCP guidance are under development and when completed will be posted in the "Continuously Reinforced Concrete Pavement Design Guide" on the Department Pavement website.

621.3 Precast Panel Concrete Pavement (PPCP)

PPCPs use panels that are precast off-site instead of cast-in-place. The precast panels can be linked together with dowel bars and tie bars or can be post-tensioned after placement. PPCP offers the advantages of:

- Improved concrete mixing and curing in a precast yard.
- Reduced pavement thicknesses, which is beneficial when there are profile grade restrictions such as vertical clearances.
- Shorter lane closure times, which is beneficial when there are short construction windows.

The primary disadvantage of PPCP is the high cost of precasting. PPCP also needs a smoothly leveled base underneath the precast panels during construction to even out the loads on the slab and avoid uneven deflection that could lead to faulting at the joints, slab settlement, and premature cracking. PPCP is currently used on an experimental basis in California, and must follow the procedures for experimental projects and special designs discussed in Topic 606.

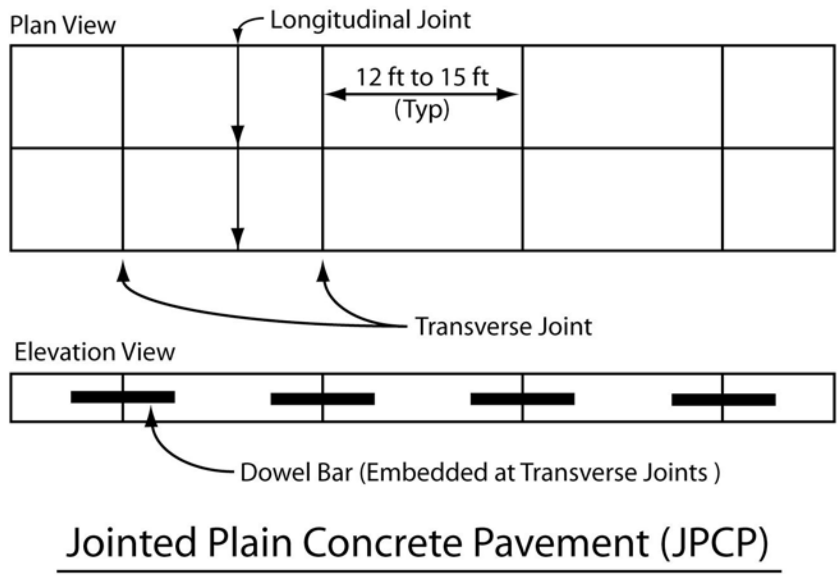
Topic 622 - Engineering Requirements

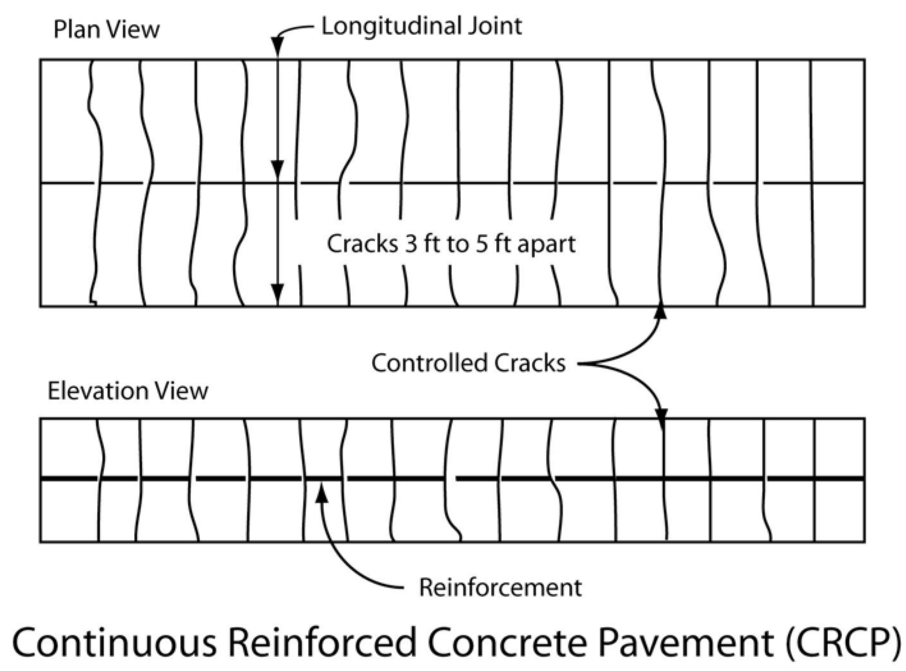
622.1 Engineering Properties

Table 622.1 shows the rigid pavement engineering properties that were used to develop the rigid pavement catalog in Index 623.1. The values are based on Department specifications and experience with materials used in California. The predominant type of concrete used in California for rigid pavement is Portland cement concrete. Other types of hydraulic cement concrete are sometimes used for special conditions such as rapid strength concrete.

Figure 621.1

Types of Rigid Pavement





(1) *Smoothness.* The smoothness of a pavement impacts its ride quality, overall durability, and performance. Ride quality (measured by the smoothness of ride) is also the highest concern listed in public surveys on pavement condition. Smoothness specifications have been improved and incentive/disincentive specifications have been developed to assure that smoothness values are achieved in construction. Incentive/disincentive specifications can be used where the project meets the warrants for the smoothness specification. For up to date, additional information on smoothness and application of specifications see the smoothness page on the Department Pavement website.

622.2 Performance Factors

The performance factors used to engineer rigid pavements are shown in Table 622.2. The pavement structure in Index 623.1 is expected to meet or exceed all of the performance factors in Table 622.2. The performance factors in the table are end-of-design life criteria.

622.3 Pavement Joints

(1) *Construction.* Construction joints (sometimes called contact or cold joint) are joints between slabs that result when concrete is placed at different times. Construction joints can be transverse or longitudinal and are constructed in all types of rigid pavements. Tie bars are typically used at construction joints to connect the adjoining slabs together so that the construction joint will be tightly closed.

(2) *Contraction.* Longitudinal and transverse contraction joints (also known as weakened plane joints) are sawed into new pavement to control the location and geometry of shrinkage, curling, and thermal cracking.

(3) *Isolation.* Isolation joints are used to separate dissimilar pavements/structures in order to lessen compressive stresses that could cause excessive cracking. Examples of dissimilar pavements/structures include different joint patterns, different types of rigid pavement (e.g., CRCP/JPCP), structure approach slabs, building foundations, drainage inlets, and manholes. Isolation joints are filled with a joint filler material to keep cracks from propagating through the joint and to prevent water/dirt infiltration.

(4) *Expansion.* Expansion joints (known previously as pressure relief joints) are similar in purpose to isolation joints except they are used where there is a need to allow for a large expansion, greater than ½ inch, between slabs or pavements. Expansion joints are typically used where CRCP abuts up to bridges, structure approach slabs or other types of rigid pavements. Expansion joints are also used with PPCP. Expansion joints are typically not used with JPCP.

Additional information on rigid pavement joints and when, where, and how to place them can be found in the Standard Plans, Standard Specifications/Special Provisions, Pavement Interactive Guide, and the Department Pavement website.

622.4 Dowel Bars and Tie Bars

Dowel bars are smooth round bars that act as load transfer devices across pavement joints. Dowel bars are typically placed across transverse joints of jointed plain and precast panel concrete pavement. In limited situations, dowel bars are placed across longitudinal joints. See Standard Plans for further details. Tie bars are deformed bars (i.e., rebar) or connectors that are used to hold the faces of abutting rigid slabs in contact. Tie bars are typically placed across longitudinal joints. Further details regarding dowel bars and tie bars can be found in the Standard Plans and Pavement Technical Guidance on the Department Pavement website.

New or reconstructed rigid pavements and lane replacements shall be doweled except as noted below:

- Rigid shoulders placed or reconstructed next to a nondoweled rigid lane may be nondoweled.
- Rigid shoulders placed or reconstructed next to a widened slab may be nondoweled and untied (see Standard Plan P-2).

Table 622.1

Rigid Pavement Engineering Properties

Property	Values
Transverse joint spacing	13.5 ft average
Initial IRI immediately after construction	63 in/mile max
Reliability	90%
Unit weight	150 lb/ft^3
Poisson's ratio	0.20
Coefficient of thermal expansion	6.0 x 10^{-6}/ °F
Thermal conductivity	$1.25 \frac{\text{Btu}}{\text{hr - ft - °F}}$
Heat capacity	$0.28 \frac{\text{Btu}}{\text{lbm - °F}}$
Permanent curl/warp effective temperature difference	Top of slab is 10 °F cooler than bottom of slab
Surface layer/base interface	Unbonded
Surface shortwave absorptivity	0.85
Cement type	Type II Portland Cement
Cement material content (cement + flyash)	24 lb/ft^3
Water: cementitious material ratio	0.42
PCC zero-stress temperature	100.9 °F
Ultimate shrinkage at 40% relative humidity	537 microstrain
Reversible shrinkage (% of ultimate shrinkage)	50%
Time to develop ultimate shrinkage	35 days
Modulus of rupture or flexural strength (28 days)	625 psi
Dowel bar diameter	1.5 in (1.25 in for rigid pavement thickness < 0.70 ft)

Table 622.2

Rigid Pavement Performance Factors

Factor	Value
General	
Design Life	Determined per Topic 612
Terminal IRI [1] at end of design life	160 in/mile max
JPCP only	
Transverse cracking at end of design life	10% of slabs max
Longitudinal cracking at end of design life	10% of slabs max
Corner cracking at end of design life	10% of slabs max
Average joint faulting at end of design life	0.10 inch max
CRCP only	
Punchouts at end of design life	10 per mile max

NOTE:

(1) The International Roughness Index (IRI) is a nationally recognized method for measuring the smoothness of pavements.

- Rigid pavement should not be tied to adjacent rigid pavement when the spacing of transverse joints of adjacent slabs is not the same.
- No more than 50 feet width of rigid pavement should be tied together to preclude random longitudinal cracks from occurring due to the pavement acting as one large rigid slab. In order to maintain some load transfer across the longitudinal joint, Standard Plan P18 includes details for placing dowel bars in the longitudinal joint for this situation.

For individual slab replacements, the placement of dowel bars is determined on a project-by-project basis based on proposed design life, construction work windows, existence of dowel bars in adjacent slabs, condition of adjacent slabs, and other pertinent factors. For further information on slab replacements, see Standard Plan P8, the "Slab Replacement Guide" and supplementary "Design Tools for Slab and Lane Replacements" on the Department Pavement website.

622.5 Joint Seals

(1) *General.* Joint and crack seals are used to protect wide joints (joints 3/8 inch or wider) from infiltration of surface moisture and intrusion of incompressible materials. Infiltration of surface moisture and intrusion of incompressible materials into joints is minimized when a narrow joint is used.

(2) *New Construction, Widening, and Reconstruction.* Joints are not sealed for new construction, widening, or for reconstruction except for the following conditions:

- isolation joints,
- expansion joints,
- longitudinal construction joints in all desert and mountain climate regions, and
- transverse joints in JPCP in all desert and mountain climate regions.

(3) *Preservation and Rehabilitation.* To be effective, existing joint seals should be replaced every 10 to 15 years depending on the type used. As part of preservation or rehabilitation strategies, existing joint seals should be replaced when the pavement is ground, replaced or dowel bar retrofitted. Previously unsealed joints should be reviewed to determine if joint sealing is warranted in accordance with the criteria in the Maintenance Technical Advisory Guide. The condition of the existing joints and joint seals should be reviewed with the District Maintenance Engineer to determine if joint seal replacement is warranted.

(4) *Selection of Joint Seal Material.* Various products are available for sealing joints with each one differing in cost and service life.

The type of joint sealant is selected based on the following criteria:

- Project environment.

  In mountain and high desert climate regions where chains are used during winter storms, joint sealants that use backer rods are not recommended. Severe climate conditions (such as in the mountains or deserts) will require more durable sealants and/or more frequent replacement.

- Type of roadway.

  Interstate or State highway, and corresponding traffic characteristics including traffic volumes and percentage of truck traffic.

- Condition of existing reservoir.

  If the sides of in-place joint faces are variable in condition, do not use preformed compression seal.

- Expected performance.

  If suitable for intended use and site conditions, the sealant with the longest service life is preferred.

  The joint sealant selected should match the type of existing joint sealant being left in place.

- Cost effectiveness.

  Life cycle cost analysis (LCCA) is used to select the appropriate sealant type.

  Joint sealants should not last longer than the pavement being sealed.

For additional information on various joint seal products and selection guidance, consult the Maintenance Technical Advisory Guide on the Department Pavement website.

622.6 Bond Breaker

When placing rigid pavement over a lean concrete base, it is important to avoid bonding between the two layers. Bonding can cause cracks and joints in the lean concrete base to reflect through the rigid pavement, which will lead to premature cracking. Several methods are available for preventing bonding including a liberal application of wax curing compound, or slurry seals. Application rates may be found in the Standard Specifications. For specific recommendations on how to prevent bonding between rigid pavement and lean concrete base, consult the District Materials Engineer.

622.7 Texturing

Longitudinal tining is the typical texturing for new pavements. Grooving is typically done to rehabilitate existing pavement texture or to improve surface friction. Grinding is typically done to restore a smooth riding surface on existing pavements or for individual slab replacements. Grooving or grinding are options on new pavement in lieu of longitudinal tining where there is a desire to minimize noise levels on rigid pavement.

622.8 Transitions and Anchors

Transitions and anchors are used at transverse joints to minimize deterioration or faulting of the joint where rigid pavement abuts to flexible pavement, a different rigid pavement type, or in some cases, a bridge. For JPCP, a pavement end anchor or transition should be used at transitions to flexible pavement. **For CRCP, a terminal anchor or terminal joint shall be used at all transitions to or from structure approach slabs, JPCP, PPCP, or flexible pavement.** Standard Plans include a variety of details for these transitions.

Topic 623 - Engineering Procedure for New and Reconstruction Projects

623.1 Catalog

Tables 623.1B through M contain the minimum thickness for rigid pavement surface layers, base, and subbase for all types of projects. All JPCP structures shown are doweled. The tables are categorized by subgrade soil type and climate regions. Figure 623.1 is used to determine which table to use to select the pavement structure.

The steps for selecting the appropriate rigid pavement structure are as follows:

(1) *Determine the Soil Type for the Existing Subgrade*. Soil types for existing subgrade are

categorized into Types I, II, and III as shown in Table 623.1A. Soils are classified by the Unified Soil Classification System (USCS). If a soil can be classified in more than one type in Table 623.1A, then the engineer should choose the more conservative design based on the less stable soil. Subgrade is discussed in Topic 614.

(2) *Determine Climate Region.* Find the location of the project on the Pavement Climate Map. The Pavement Climate Map is discussed in Topic 615.

(3) *Select the Appropriate Table (Tables 623.1B through M).* Select the table that applies to the project based on subgrade, soil type, and climate region. Use Figure 623.1 to determine which table applies to the project.

(4) *Determine Whether Pavement Has Lateral Support Along Both Longitudinal Joints.* The pavement is considered laterally supported if it is tied to an adjacent lane, has tied rigid shoulders, or has a widened slab. If lateral support is provided along only one longitudinal joint, then the pavement is considered to have no lateral support. As shown in Tables 623.1B through M, pavement thicknesses are reduced slightly for slabs engineered with lateral support along both longitudinal joints.

(5) *Select Pavement Structure.* Using the Traffic Index provided or calculated from the traffic projections, select the desired pavement structure from the list of alternatives provided.

Note that although the pavement structures listed for each Traffic Index are considered to be acceptable for the climate, soil conditions, and design life desired, they should not be considered as equal designs. Some designs will perform better than others, have lower maintenance/repair costs, and/or lower construction life-cycle costs. Sound engineering judgment should be used in selecting the option that is most effective for the location. For these reasons, the rigid pavement structures in these tables cannot be used as substitutes for the pavement structures recommended in approved Materials Reports or shown in approved contract plans.

Table 623.1A

Relationship Between Subgrade Type[(1)]

Subgrade Type[(2)]	California R-value (R)	Unified Soil Classification System (USCS)
I	R > 40	SC, SP, SM, SW, GC, GP, GM, GW
II	$10 \le R \le 40$	CH (PI $\le$ 12), CL, MH, ML
III	R < 10	CH (PI >12)

NOTES:

(1) See Topic 614 for further discussion on subgrade and USCS.

(2) Choose more conservative soil type (i.e., use soil with a lower R-value or USCS) if native soil can be classified by more than one type.

Legend

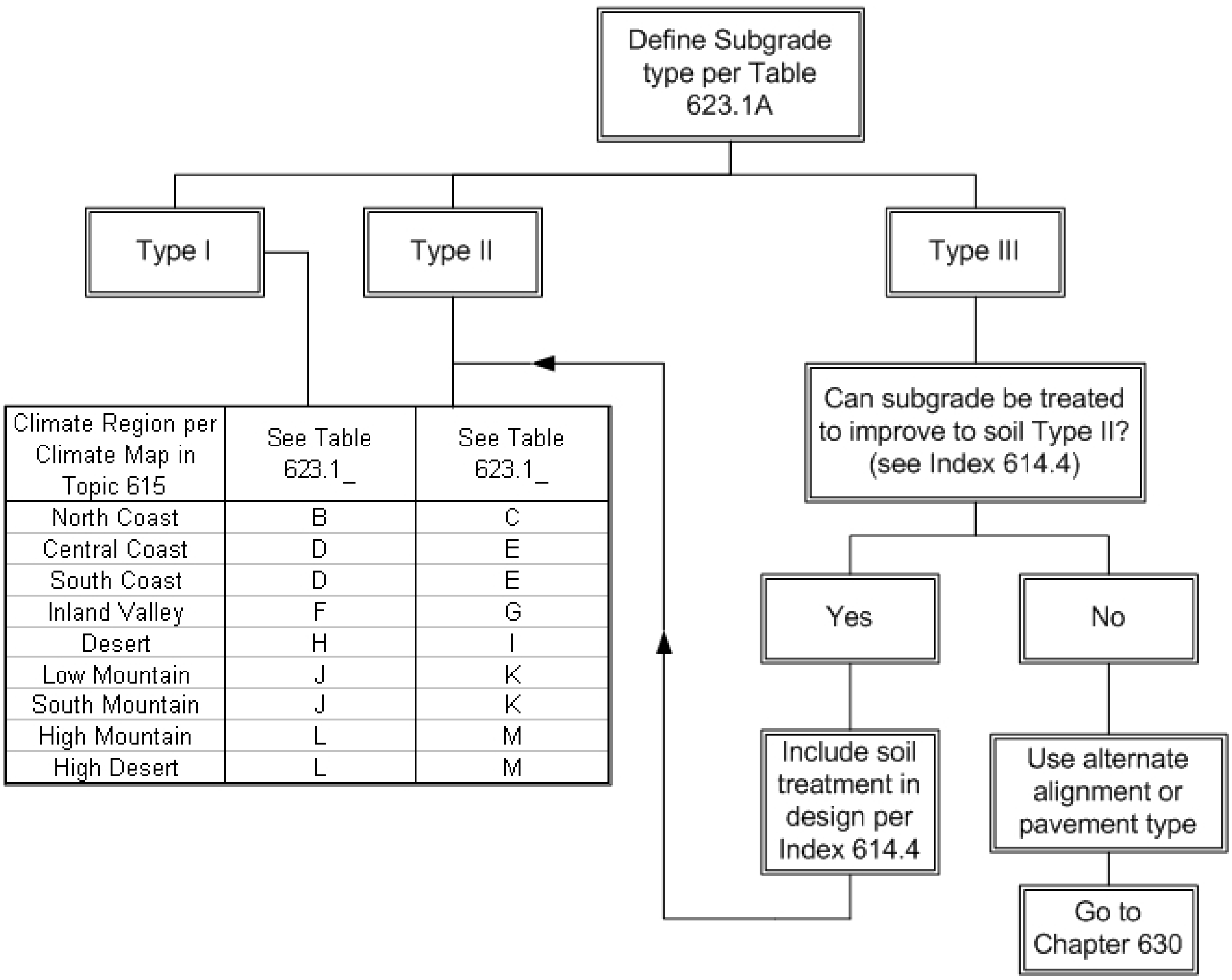
PI = Plasticity Index

623.2 Mechanistic-Empirical Method

For information on Mechanistic-Empirical Design application and requirements, see Index 606.3.

Figure 623.1

Rigid Pavement Catalog Decision Tree



Climate Region per Climate Map in Topic 615	See Table 623.1_	See Table 623.1_
North Coast	B	C
Central Coast	D	E
South Coast	D	E
Inland Valley	F	G
Desert	H	I
Low Mountain	J	K
South Mountain	J	K
High Mountain	L	M
High Desert	L	M

Table 623.1B
Rigid Pavement Catalog (North Coast, Type I Subgrade Soil)(1), (2), (3), (4),(5)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.70 JPCP 0.50 AB	0.70 JPCP 0.35 ATPB 0.35 AB	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.70 JPCP 0.50 AB	0.70 JPCP 0.35 ATPB 0.35 AB
9.5 to 10	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.75 JPCP 0.60 AB	0.75 JPCP 0.35 ATPB 0.40 AB	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25HMA-A	0.75 JPCP 0.60 AB	0.75 JPCP 0.35 ATPB 0.40 AB
10.5 to 11	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.70 JPCP 0.70 AB		0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.75 JPCP 0.70 AB	
11.5 to 12	0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.75 CRCP 0.35 HMA-A		0.80 JPCP 0.35 LCB	0.80 JPCP 0.25HMA-A	0.80 CRCP 0.40 HMA-A	
12.5 to 13	0.80 JPCP 0.35 LCB	0.80 JPCP 0.25 HMA-A	0.75 CRCP 0.25 HMA-A		0.85 JPCP 0.50 LCB	0.85 JPCP 0.50 HMA-A	0.80 CRCP 0.50 HMA-A	
13.5 to 14	0.80 JPCP 0.35 LCB	0.80 JPCP 0.25 HMA-A	0.75 CRCP 0.25 HMA-A		0.90 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A	
14.5 to 15	0.85 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A		0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A	
15.5 to 16	0.90 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.00 JPCP 0.35 LCB	1.00 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A	
16.5 to 17	0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.05 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A	
> 17 < 9	1.00 JPCP 0.35 LCB 0.70 JPCP	1.00 JPCP 0.25 HMA-A 0.70 JPCP	0.90 CRCP 0.25 HMA-A 0.70 JPCP	0.70 JPCP	1.10 JPCP 0.35LCB 0.70 JPCP	1.10 JPCP 0.25 HMA-A 0.70 JPCP	1.00 CRCP 0.25 HMA-A 0.70 JPCP	0.70 JPCP

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Table 623.1C
Rigid Pavement Catalog (North Coast, Type II Subgrade Soil) (1), (2), (3), (4), (5)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
≤ 9	0.70 JPCP 0.35 LCB 0.50 AS	0.70 JPCP 0.25 HMA-A 0.50 AS	0.70 JPCP 1.00 AB	0.70 JPCP 0.35 ATPB 0.80 AB	0.70 JPCP 0.35 LCB 0.50 AS	0.70 JPCP 0.25 HMA-A 0.50 AS	0.70 JPCP 1.00 AB	0.70 JPCP 0.35 ATPB 0.80 AB
9.5 to 10	0.70 JPCP 0.35 LCB 0.50 AS	0.70 JPCP 0.25 HMA-A 0.50 AS	0.75 JPCP 1.00 AB	0.75 JPCP 0.35 ATPB 0.80 AB	0.70 JPCP 0.35 LCB 0.50 AS	0.70 JPCP 0.25 HMA-A 0.50 AS	0.75 JPCP 1.00 AB	0.75 JPCP 0.35 ATPB 0.80 AB
10.5 to 11	0.70 JPCP 0.35 LCB 0.60 AS	0.70 JPCP 0.25 HMA-A 0.60 AS	0.70 JPCP 1.30 AB		0.75 JPCP 0.35 LCB 0.60 AS	0.75 JPCP 0.25 HMA-A 0.60 AS	0.75 JPCP 1.30 AB	
11.5 to 12	0.75 JPCP 0.35 LCB 0.60 AS	0.75 JPCP 0.25 HMA-A 0.60 AS	0.75 CRCP 0.25 HMA-A 0.60 AS		0.80 JPCP 0.35 LCB 0.60 AS	0.80 JPCP 0.25 HMA-A 0.60 AS	0.80 CRCP 0.25 HMA-A 0.60 AS	
12.5 to 13	0.80 JPCP 0.35 LCB 0.70 AS	0.80 JPCP 0.25 HMA-A 0.70 AS	0.75 CRCP 0.25 HMA-A 0.70 AS		0.85 JPCP 0.35 LCB 0.70 AS	0.85 JPCP 0.25 HMA-A 0.70 AS	0.80 CRCP 0.25 HMA-A 0.70 AS	
13.5 to 14	0.80 JPCP 0.35 LCB 0.70 AS	0.80 JPCP 0.25 HMA-A 0.70 AS	0.75 CRCP 0.25 HMA-A 0.70 AS		0.90 JPCP 0.35 LCB 0.70 AS	0.85 JPCP 0.25 HMA-A 0.70 AS	0.80 CRCP 0.25 HMA-A 0.70 AS	
14.5 to 15	0.85 JPCP 0.35 LCB 0.70 AS	0.85 JPCP 0.25 HMA-A 0.70 AS	0.80 CRCP 0.25 HMA-A 0.70 AS		0.95 JPCP 0.35 LCB 0.70 AS	0.95 JPCP 0.25 HMA-A 0.70 AS	0.85 CRCP 0.25 HMA-A 0.70 AS	
15.5 to 16	0.90 JPCP 0.35 LCB 0.70 AS	0.90 JPCP 0.25HMA-A 0.70 AS	0.85 CRCP 0.25 HMA-A 0.70 AS		1.00 JPCP 0.35 LCB 0.70 AS	1.00 JPCP 0.25 HMA-A 0.70 AS	0.90 CRCP 0.25 HMA-A 0.70 AS	
16.5 to 17	0.95 JPCP 0.35 LCB 0.70 AS	0.95 JPCP 0.25 HMA-A 0.70 AS	0.85 CRCP 0.25 HMA-A 0.70 AS		1.05 JPCP 0.35 LCB 0.70 AS	1.05 JPCP 0.25 HMA-A 0.70 AS	0.95 CRCP 0.25 HMA-A 0.70 AS	
> 17	1.00 JPCP 0.35 LCB 0.70 AS	1.00 JPCP 0.25 HMA-A 0.70 AS	0.90 CRCP 0.25 HMA-A 0.70 AS		1.10 JPCP 0.35 LCB 0.70 AS	1.10 JPCP 0.25 HMA-A 0.70 AS	1.00 CRCP 0.25 HMA-A 0.70 AS	

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP =	Jointed Plain Concrete Pavement	ATPB =	Asphalt Treated Permeable Base
CRCP =	Continuously Reinforced Concrete Pavement	AB =	Class 2 Aggregate Base
LCB =	Lean Concrete Base	AS =	Class 2 Aggregate Subbase
HMA-A =	Hot Mix Asphalt (Type A)	TI =	Traffic Index

Table 623.1D
Rigid Pavement Catalog
(South Coast/Central Coast, Type I Subgrade Soil) (1), (2), (3), (4), (5)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.70 JPCP 0.50 AB	0.70 JPCP 0.35 ATPB 0.35 AB	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.75 JPCP 0.50 AB	0.75 JPCP 0.35 ATPB 0.35 AB
9.5 to 10	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.75 JPCP 0.60 AB	0.75 JPCP 0.35 ATPB 0.40 AB	0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.80 JPCP 0.60 AB	0.80 JPCP 0.35 ATPB 0.40 AB
10.5 to 11	0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.80 JPCP 0.70 AB		0.80 JPCP 0.35 LCB	0.80 JPCP 0.25 HMA-A	0.85 JPCP 0.70 AB	
11.5 to 12	0.80 JPCP 0.35 LCB	0.80 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A		0.85 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A	
12.5 to 13	0.85 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A		0.90 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A	
13.5 to 14	0.85 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A		0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A	
14.5 to 15	0.90 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.00 JPCP 0.35 LCB	1.00 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A	
15.5 to 16	0.95 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.05 JPCP 0.35LCB	1.05 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A	
16.5 to 17	1.00 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A		1.10 JPCP 0.35 LCB	1.10 JPCP 0.25 HMA-A	1.00 CRCP 0.25 HMA-A	
> 17	1.05 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A		1.15 JPCP 0.35 LCB	1.15 JPCP 0.25 HMA-A	1.00 CRCP 0.25 HMA-A	

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Table 623.1E
Rigid Pavement Catalog
(South Coast/Central Coast, Type II Subgrade Soil) (1), (2), (3), (4), (5)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.75 JPCP	0.75 JPCP
< 9	0.35 LCB	0.25 HMA-A	1.00 AB	0.35 ATPB	0.35 LCB	0.25 HMA-A	1.00 AB	0.35 ATPB
	0.50 AS	0.50 AS		0.80 AB	0.50 AS	0.50 AS		0.80 AB
	0.70 JPCP	0.70 JPCP	0.75 JPCP	0.75 JPCP	0.75 JPCP	0.75 JPCP	0.80 JPCP	0.80 JPCP
9.5 to 10	0.35 LCB	0.25 HMA-A	1.00 AB	0.35 ATPB	0.35 LCB	0.25 HMA-A	1.00 AB	0.35 ATPB
	0.50 AS	0.50 AS		0.80 AB	0.50 AS	0.50 AS		0.80 AB
	0.75 JPCP	0.75 JPCP	0.80 JPCP		0.80 JPCP	0.80 JPCP	0.85 JPCP	
10.5 to 11	0.35 LCB	0.25 HMA-A	1.30 AB		0.35 LCB	0.25 HMA-A	1.30 AB	
	0.60 AS	0.60 AS			0.60 AS	0.60 AS		
	0.80 JPCP	0.80 JPCP	0.80 CRCP		0.85 JPCP	0.85 JPCP	0.80 CRCP	
11.5 to 12	0.35 LCB	0.25 HMA-A	0.25 HMA-A		0.35 LCB	0.25 HMA-A	0.25 HMA-A	
	0.60 AS	0.60 AS	0.60 AS		0.60 AS	0.60 AS	0.60 AS	
	0.85 JPCP	0.85 JPCP	0.80 CRCP		0.90 JPCP	0.90 JPCP	0.85 CRCP	
12.5 to 13	0.35 LCB	0.25 HMA-A	0.25 HMA-A		0.35 LCB	0.25 HMA-A	0.25 HMA-A	
	0.70 AS	0.70 AS	0.70 AS		0.70 AS	0.70 AS	0.70 AS	
	0.85 JPCP	0.85 JPCP	0.80 CRCP		0.95 JPCP	0.95 JPCP	0.90 CRCP	
13.5 to 14	0.35 LCB	0.25 HMA-A	0.25 HMA-A		0.35 LCB	0.25 HMA-A	0.25 HMA-A	
	0.70 AS	0.70 AS	0.70 AS		0.70 AS	0.70 AS	0.70 AS	
	0.90 JPCP	0.90 JPCP	0.85 CRCP		1.00 JPCP	1.00 JPCP	0.95 CRCP	
14.5 to 15	0.35 LCB	0.25 HMA-A	0.25 HMA-A		0.35 LCB	0.25 HMA-A	0.25 HMA-A	
	0.70 AS	0.70 AS	0.70 AS		0.70 AS	0.70 AS	0.70 AS	
	0.95 JPCP	0.90 JPCP	0.85 CRCP		1.05 JPCP	1.05 JPCP	0.95 CRCP	
15.5 to 16	0.35 LCB	0.25 HMA-A	0.25 HMA-A		0.35 LCB	0.25 HMA-A	0.25 HMA-A	
	0.70 AS	0.70 AS	0.70 AS		0.70 AS	0.70 AS	0.70 AS	
	1.00 JPCP	0.95 JPCP	0.90 CRCP		1.10 JPCP	1.10 JPCP	1.00 CRCP	
16.5 to 17	0.35 LCB	0.25 HMA-A	0.25 HMA-A		0.35 LCB	0.25 HMA-A	0.25 HMA-A	
	0.70 AS	0.70 AS	0.70 AS		0.70 AS	0.70 AS	0.70 AS	
	1.05 JPCP	1.05 JPCP	0.95 CRCP		1.15 JPCP	1.15 JPCP	1.00 CRCP	
> 17	0.35 LCB	0.25 HMA-A	0.25 HMA-A		0.35 LCB	0.25 HMA-A	0.25 HMA-A	
	0.70 AS	0.70 AS	0.70 AS		0.70 AS	0.70 AS	0.70 AS	

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
AS = Class 2 Aggregate Subbase
TI = Traffic Index

Table 623.1F
Rigid Pavement Catalog (Inland Valley, Type I Subgrade Soil) (1), (2), (3), (4), (5)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.75 JPCP 0.50 AB	0.70 JPCP 0.35 ATPB 0.35 AB	0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.80 JPCP 0.50 AB	0.75 JPCP 0.35 ATPB 0.35 AB
9.5 to 10	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.80 JPCP 0.60 AB	0.75 JPCP 0.35 ATPB 0.40 AB	0.80 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.90 JPCP 0.60 AB	0.85 JPCP 0.35 ATPB 0.40 AB
10.5 to 11	0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.85 JPCP 0.70 AB		0.85 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.95 JPCP 0.70 AB	
11.5 to 12	0.85 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A		0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A	
12.5 to 13	0.85 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A		1.00 JPCP 0.35 LCB	1.00 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A	
13.5 to 14	0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.05 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A	
14.5 to 15	1.00 JPCP 0.35 LCB	1.00 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A		1.15 JPCP 0.35 LCB	1.15 JPCP 0.25 HMA-A	1.00 CRCP 0.25 HMA-A	
15.5 to 16	1.05 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A		1.20 JPCP 0.35 LCB	1.20 JPCP 0.25 HMA-A	1.05 CRCP 0.25 HMA-A	
16.5 to 17	1.10 JPCP 0.35 LCB	1.10 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A		1.25 JPCP 0.35 LCB	1.25 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A	
> 17	1.15 JPCP 0.35 LCB	1.15 JPCP 0.25 HMA-A	1.00 CRCP 0.25 HMA-A		1.30 JPCP 0.35 LCB	1.30 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A	

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Table 623.1G
Rigid Pavement Catalog (Inland Valley, Type II Subgrade Soil) (1), (2), (3), (4), (5)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.70 JPCP 0.35 LCB 0.50 AS	0.70 JPCP 0.25 HMA-A 0.50 AS	0.75 JPCP 1.00 AB	0.70 JPCP 0.35 ATPB 0.80 AB	0.75 JPCP 0.35 LCB 0.50 AS	0.75 JPCP 0.25 HMA-A 0.50 AS	0.80 JPCP 1.00 AB	0.75 JPCP 0.35 ATPB 0.80 AB
9.5 to 10	0.70 JPCP 0.35 LCB 0.50 AS	0.70 JPCP 0.25 HMA-A 0.50 AS	0.80 JPCP 1.00 AB	0.75 JPCP 0.35 ATPB 0.80 AB	0.80 JPCP 0.35 LCB 0.50 AS	0.85 JPCP 0.25 HMA-A 0.50 AS	0.90 JPCP 1.00 AB	0.85 JPCP 0.35 ATPB 0.80 AB
10.5 to 11	0.75 JPCP 0.35 LCB 0.60 AS	0.75 JPCP 0.25 HMA-A 0.60 AS	0.85 JPCP 1.30 AB		0.85 JPCP 0.35 LCB 0.60 AS	0.90 JPCP 0.25 HMA-A 0.60 AS	0.95 JPCP 1.30 AB	
11.5 to 12	0.85 JPCP 0.35 LCB 0.60 AS	0.85 JPCP 0.25 HMA-A 0.60 AS	0.80 CRCP 0.25 HMA-A 0.60 AS		0.95 JPCP 0.35 LCB 0.60 AS	0.95 JPCP 0.25 HMA-A 0.60 AS	0.85 CRCP 0.25 HMA-A 0.60 AS	
12.5 to 13	0.85 JPCP 0.35 LCB 0.70 AS	0.90 JPCP 0.25 HMA-A 0.70 AS	0.80 CRCP 0.25 HMA-A 0.70 AS		1.00 JPCP 0.35 LCB 0.70 AS	1.00 JPCP 0.25 HMA-A 0.70 AS	0.90 CRCP 0.25 HMA-A 0.70 AS	
13.5 to 14	0.95 JPCP 0.35 LCB 0.70 AS	0.95 JPCP 0.25 HMA-A 0.70 AS	0.85 CRCP 0.25 HMA-A 0.70 AS		1.05 JPCP 0.35 LCB 0.70 AS	1.05 JPCP 0.25 HMA-A 0.70 AS	0.95 CRCP 0.25 HMA-A 0.70 AS	
14.5 to 15	1.00 JPCP 0.35 LCB 0.70 AS	1.00 JPCP 0.25 HMA-A 0.70 AS	0.90 CRCP 0.25 HMA-A 0.70 AS		1.15 JPCP 0.35 LCB 0.70 AS	1.15 JPCP 0.25 HMA-A 0.70 AS	1.00 CRCP 0.25 HMA-A 0.70 AS	
15.5 to 16	1.05 JPCP 0.35 LCB 0.70 AS	1.05 JPCP 0.25HMA-A 0.70 AS	0.95 CRCP 0.25 HMA-A 0.70 AS		1.20 JPCP 0.35 LCB 0.70 AS	1.20 JPCP 0.25 HMA-A 0.70 AS	1.05 CRCP 0.25 HMA-A 0.70 AS	
16.5 to 17	1.10 JPCP 0.35 LCB 0.70 AS	1.10 JPCP 0.25 HMA-A 0.70 AS	0.95 CRCP 0.25 HMA-A 0.70 AS		1.25 JPCP 0.35 LCB 0.70 AS	1.25 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS	
> 17	1.15 JPCP 0.35 LCB 0.70 AS	1.15 JPCP 0.25 HMA-A 0.70 AS	1.00 CRCP 0.25 HMA-A 0.70 AS		1.30 JPCP 0.35 LCB 0.70 AS	1.30 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS	

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP =	Jointed Plain Concrete Pavement	ATPB =	Asphalt Treated Permeable Base
CRCP =	Continuously Reinforced Concrete Pavement	AB =	Class 2 Aggregate Base
LCB =	Lean Concrete Base	AS =	Class 2 Aggregate Subbase
HMA-A =	Hot Mix Asphalt (Type A)	TI =	Traffic Index

Table 623.1H
Rigid Pavement Catalog (Desert, Type I Subgrade Soil) (1), (2), (3), (4), (5)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.75 JPCP 0.50 AB	0.70 JPCP 0.35 ATPB 0.40 AB	0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.80 JPCP 0.50 AB	0.75 JPCP 0.35 ATPB 0.40 AB
9.5 to 10	0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.80 JPCP 0.60 AB	0.80 JPCP 0.35 ATPB 0.40 AB	0.80 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.90 JPCP 0.60 AB	0.85 JPCP 0.35 ATPB 0.40 AB
10.5 to 11	0.80 JPCP 0.35 LCB	0.80 JPCP 0.25 HMA-A	0.85 JPCP 0.70 AB		0.85 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.95 JPCP 0.70 AB	
11.5 to 12	0.85 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A		0.90 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A	
12.5 to 13	0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.05 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A	
13.5 to 14	1.00 JPCP 0.35 LCB	1.00 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A		1.15 JPCP 0.35 LCB	1.15 JPCP 0.25 HMA-A	1.05 CRCP 0.25 HMA-A	
14.5 to 15	1.05 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A		1.20 JPCP 0.35 LCB	1.20 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A	
15.5 to 16	1.10 JPCP 0.35 LCB	1.10 JPCP 0.25 HMA-A	1.00 CRCP 0.25 HMA-A		1.25 JPCP 0.35 LCB	1.25 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A	
16.5 to 17	1.15 JPCP 0.35 LCB	1.15 JPCP 0.25 HMA-A	1.05 CRCP 0.25 HMA-A		1.30 JPCP 0.35 LCB	1.30 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A	
> 17	1.20 JPCP 0.35 LCB	1.20 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A		1.30 JPCP 0.35 LCB	1.30 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A	

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Table 623.1I
Rigid Pavement Catalog (Desert, Type II Subgrade Soil) (1), (2), (3), (4), (5)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.70 JPCP 0.35 LCB 0.50 AS	0.70 JPCP 0.25 HMA-A 0.50 AS	0.75 JPCP 1.00 AB	0.70 JPCP 0.35 ATPB 0.80 AB	0.75 JPCP 0.35 LCB 0.60 AS	0.75 JPCP 0.25 HMA-A 0.60 AS	0.80 JPCP 1.00 AB	0.75 JPCP 0.35 ATPB 0.80 AB
9.5 to 10	0.75 JPCP 0.35 LCB 0.50 AS	0.75 JPCP 0.25 HMA-A 0.50 AS	0.80 JPCP 1.00 AB	0.80 JPCP 0.35 ATPB 0.80 AB	0.80 JPCP 0.35 LCB 0.60 AS	0.85 JPCP 0.25 HMA-A 0.60 AS	0.90 JPCP 1.00 AB	0.85 JPCP 0.35 ATPB 0.80 AB
10.5 to 11	0.80 JPCP 0.35 LCB 0.60 AS	0.80 JPCP 0.25 HMA-A 0.60 AS	0.85 JPCP 1.30 AB		0.85 JPCP 0.35 LCB 0.60 AS	0.90 JPCP 0.25 HMA-A 0.60 AS	0.95 JPCP 1.30 AB	
11.5 to 12	0.85 JPCP 0.35 LCB 0.60 AS	0.85 JPCP 0.25 HMA-A 0.60 AS	0.80 CRCP 0.25 HMA-A 0.60 AS		0.90 JPCP 0.35 LCB 0.60 AS	0.95 JPCP 0.25 HMA-A 0.60 AS	0.85 CRCP 0.25 HMA-A 0.60 AS	
12.5 to 13	0.95 JPCP 0.35 LCB 0.70 AS	0.95 JPCP 0.25 HMA-A 0.70 AS	0.85 CRCP 0.25 HMA-A 0.70 AS		1.05 JPCP 0.35 LCB 0.70 AS	1.05 JPCP 0.25 HMA-A 0.70 AS	0.95 CRCP 0.25 HMA-A 0.70 AS	
13.5 to 14	1.00 JPCP 0.35 LCB 0.70 AS	1.00 JPCP 0.25 HMA-A 0.70 AS	0.90 CRCP 0.25 HMA-A 0.70 AS		1.15 JPCP 0.35 LCB 0.70 AS	1.15 JPCP 0.25 HMA-A 0.70 AS	1.05 CRCP 0.25 HMA-A 0.70 AS	
14.5 to 15	1.05 JPCP 0.35 LCB 0.70 AS	1.05 JPCP 0.25 HMA-A 0.70 AS	0.95 CRCP 0.25 HMA-A 0.70 AS		1.20 JPCP 0.35 LCB 0.70 AS	1.20 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS	
15.5 to 16	1.10 JPCP 0.35 LCB 0.70 AS	1.10 JPCP 0.25 HMA-A 0.70 AS	1.00 CRCP 0.25 HMA-A 0.70 AS		1.25 JPCP 0.35 LCB 0.70 AS	1.25 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS	
16.5 to 17	1.15 JPCP 0.35 LCB 0.70 AS	1.15 JPCP 0.25 HMA-A 0.70 AS	1.05 CRCP 0.25 HMA-A 0.70 AS		1.30 JPCP 0.35 LCB 0.70 AS	1.30 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS	
> 17	1.20 JPCP 0.35 LCB 0.70 AS	1.20 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS		1.30 JPCP 0.35 LCB 0.70 AS	1.30 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS	

NOTES:

(1) Thicknesses shown are for doweled JPCP only. Not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
AS = Class 2 Aggregate Subbase
TI = Traffic Index

Table 623.1J
Rigid Pavement Catalog
(Low Mountain/South Mountain, Type I Subgrade Soil) (1), (2), (3), (4), (5)

TI	Rigid Pavement Structural Depth: With Lateral Support (ft)				Rigid Pavement Structural Depth: Without Lateral Support (ft)			
< 9	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.75 JPCP 0.50 AB	0.70 JPCP 0.35 ATPB 0.40 AB	0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.75 JPCP 0.50 AB	0.75 JPCP 0.35 ATPB 0.40 AB
9.5 to 10	0.70 JPCP 0.35 LCB	0.70 JPCP 0.25 HMA-A	0.75 JPCP 0.60 AB	0.75 JPCP 0.35 ATPB 0.40 AB	0.80 JPCP 0.35 LCB	0.80 JPCP 0.25 HMA-A	0.85 JPCP 0.60 AB	0.80 JPCP 0.35 ATPB 0.40 AB
10.5 to 11	0.75 JPCP 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.80 JPCP 0.70 AB		0.85 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.90 JPCP 0.70 AB	
11.5 to 12	0.80 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A		0.90 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A	
12.5 to 13	0.90 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.00 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A	
13.5 to 14	0.95 JPCP 0.35 LCB	1.00 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A		1.05 JPCP 0.35 LCB	1.10 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A	
14.5 to 15	1.00 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A		1.15 JPCP 0.35 LCB	1.20 JPCP 0.25 HMA-A	1.05 CRCP 0.25 HMA-A	
15.5 to 16	1.05 JPCP 0.35 LCB	1.10 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A		1.20 JPCP 0.35 LCB	1.25 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A	
16.5 to 17	1.10 JPCP 0.35 LCB	1.15 JPCP 0.25 HMA-A	1.00 CRCP 0.25 HMA-A		1.25 JPCP 0.35 LCB	1.30 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A	
> 17	1.15 JPCP 0.35 LCB	1.20 JPCP 0.25 HMA-A	1.00 CRCP 0.25 HMA-A		1.30 JPCP 0.35 LCB	1.35 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A	

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Table 623.1K
Rigid Pavement Catalog
(Low Mountain/South Mountain, Type II Subgrade Soil) (1), (2), (3), (4), (5)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.70 JPCP 0.35 LCB 0.50 AS	0.70 JPCP 0.25 HMA-A 0.50 AS	0.75 JPCP 1.00 AB	0.70 JPCP 0.35 ATPB 0.80 AB	0.75 JPCP 0.35 LCB 0.50 AS	0.75 JPCP 0.25 HMA-A 0.50 AS	0.75 JPCP 1.00 AB	0.75 JPCP 0.35 ATPB 0.80 AB
9.5 to 10	0.70 JPCP 0.35 LCB 0.50 AS	0.70 JPCP 0.25 HMA-A 0.50 AS	0.75 JPCP 1.00 AB	0.75 JPCP 0.35 ATPB 0.80 AB	0.80 JPCP 0.35 LCB 0.50 AS	0.80 JPCP 0.25 HMA-A 0.50 AS	0.85 JPCP 1.00 AB	0.80 JPCP 0.35 ATPB 0.80 AB
10.5 to 11	0.75 JPCP 0.35 LCB 0.60 AS	0.75 JPCP 0.25 HMA-A 0.60 AS	0.80 JPCP 1.30 AB		0.85 JPCP 0.35 LCB 0.60 AS	0.85 JPCP 0.25 HMA-A 0.60 AS	0.90 JPCP 1.30 AB	
11.5 to 12	0.80 JPCP 0.35 LCB 0.60 AS	0.85 JPCP 0.25 HMA-A 0.60 AS	0.80 CRCP 0.25 HMA-A 0.60 AS		0.90 JPCP 0.35 LCB 0.60 AS	0.95 JPCP 0.25 HMA-A 0.60 AS	0.85 CRCP 0.25 HMA-A 0.60 AS	
12.5 to 13	0.90 JPCP 0.35 LCB 0.70 AS	0.95 JPCP 0.25 HMA-A 0.70 AS	0.85 CRCP 0.25 HMA-A 0.70 AS		1.00 JPCP 0.35 LCB 0.70 AS	1.05 JPCP 0.25 HMA-A 0.70 AS	0.90 CRCP 0.25 HMA-A 0.70 AS	
13.5 to 14	0.95 JPCP 0.35 LCB 0.70 AS	1.00 JPCP 0.25 HMA-A 0.70 AS	0.85 CRCP 0.25 HMA-A 0.70 AS		1.05 JPCP 0.35 LCB 0.70 AS	1.10 JPCP 0.25 HMA-A 0.70 AS	0.95 CRCP 0.25 HMA-A 0.70 AS	
14.5 to 15	1.00 JPCP 0.35 LCB 0.70 AS	1.05 JPCP 0.25 HMA-A 0.70 AS	0.90 CRCP 0.25 HMA-A 0.70 AS		1.15 JPCP 0.35 LCB 0.70 AS	1.20 JPCP 0.25 HMA-A 0.70 AS	1.05 CRCP 0.25 HMA-A 0.70 AS	
15.5 to 16	1.05 JPCP 0.35 LCB 0.70 AS	1.10 JPCP 0.25 HMA-A 0.70 AS	0.95 CRCP 0.25 HMA-A 0.70 AS		1.20 JPCP 0.35 LCB 0.70 AS	1.25 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS	
16.5 to 17	1.10 JPCP 0.35 LCB 0.70 AS	1.15 JPCP 0.25 HMA-A 0.70 AS	1.00 CRCP 0.25 HMA-A 0.70 AS		1.25 JPCP 0.35 LCB 0.70 AS	1.30 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS	
> 17	1.15 JPCP 0.35 LCB 0.70 AS	1.20 JPCP 0.25 HMA-A 0.70 AS	1.00 CRCP 0.25 HMA-A 0.70 AS		1.30 JPCP 0.35 LCB 0.70 AS	1.35 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS	

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
AS = Class 2 Aggregate Subbase
TI = Traffic Index

Table 623.1L
Rigid Pavement Catalog
(High Mountain/High Desert, Type I Subgrade Soil) (1), (2), (3), (4), (5)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.80 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.85 JPCP 0.50 AB	0.80 JPCP 0.35 ATPB 0.40 AB	0.85 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.90 JPCP 0.50 AB	0.90 JPCP 0.35 ATPB 0.40 AB
9.5 to 10	0.85 JPCP 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.90 JPCP 0.60 AB	0.90 JPCP 0.35 ATPB 0.40 AB	0.90 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.95 JPCP 0.60 AB	0.90 JPCP 0.35 ATPB 0.40 AB
10.5 to 11	0.90 JPCP 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.95 JPCP 0.70 AB		0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A	1.00 JPCP 0.70 AB	
11.5 to 12	0.95 JPCP 0.35 LCB	0.95 JPCP 0.25 HMA-A			1.05 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A		
12.5 to 13	1.00 JPCP 0.35 LCB	1.05 JPCP 0.25 HMA-A			1.10 JPCP 0.35 LCB	1.15 JPCP 0.25 HMA-A		
13.5 to 14	1.05 JPCP 0.35 LCB	1.10 JPCP 0.25 HMA-A			1.15 JPCP 0.35 LCB	1.20 JPCP 0.25 HMA-A		
14.5 to 15	1.10 JPCP 0.35 LCB	1.15 JPCP 0.25 HMA-A			1.20 JPCP 0.35 LCB	1.25 JPCP 0.25 HMA-A		
15.5 to 16	1.15 JPCP 0.35 LCB	1.20 JPCP 0.25 HMA-A			1.25 JPCP 0.35 LCB	1.30 JPCP 0.25 HMA-A		
16.5 to 17	1.20 JPCP 0.35 LCB	1.25 JPCP 0.25 HMA-A			1.30 JPCP 0.35 LCB	1.35 JPCP 0.25 HMA-A		
> 17	1.25 JPCP 0.35 LCB	1.25 JPCP 0.25 HMA-A			1.35 JPCP 0.35 LCB	1.35 JPCP 0.25 HMA-A		

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.15 ft sacrificial wearing course for future grinding of JPCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Table 623.1M
Rigid Pavement Catalog
(High Mountain/High Desert, Type II Subgrade Soil) (1), (2), (3), (4), (5)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.80 JPCP 0.35 LCB 0.50 AS	0.85 JPCP 0.25 HMA-A 0.50 AS	0.85 JPCP 1.00 AB	0.80 JPCP 0.35 ATPB 0.80 AB	0.85 JPCP 0.35 LCB 0.50 AS	0.90 JPCP 0.25 HMA-A 0.50 AS	0.90 JPCP 1.00 AB	0.90 JPCP 0.35 ATPB 0.80 AB
9.5 to 10	0.85 JPCP 0.35 LCB 0.50 AS	0.85 JPCP 0.25 HMA-A 0.50 AS	0.90 JPCP 1.00 AB	0.90 JPCP 0.35 ATPB 0.80 AB	0.90 JPCP 0.35 LCB 0.50 AS	0.90 JPCP 0.25 HMA-A 0.50 AS	0.95 JPCP 1.00 AB	0.90 JPCP 0.35 ATPB 0.80 AB
10.5 to 11	0.90 JPCP 0.35 LCB 0.60 AS	0.90 JPCP 0.25 HMA-A 0.60 AS	0.95 JPCP 1.30 AB		0.95 JPCP 0.35 LCB 0.60 AS	0.95 JPCP 0.25 HMA-A 0.60 AS	1.00 JPCP 1.30 AB	
11.5 to 12	0.95 JPCP 0.35 LCB 0.60 AS	0.95 JPCP 0.25 HMA-A 0.60 AS			1.05 JPCP 0.35 LCB 0.60 AS	1.05 JPCP 0.25 HMA-A 0.60 AS		
12.5 to 13	1.00 JPCP 0.35 LCB 0.70 AS	1.05 JPCP 0.25 HMA-A 0.70 AS			1.10 JPCP 0.35 LCB 0.70 AS	1.15 JPCP 0.25 HMA-A 0.70 AS		
13.5 to 14	1.05 JPCP 0.35 LCB 0.70 AS	1.10 JPCP 0.25 HMA-A 0.70 AS			1.15 JPCP 0.35 LCB 0.70 AS	1.20 JPCP 0.25 HMA-A 0.70 AS		
14.5 to 15	1.10 JPCP 0.35 LCB 0.70 AS	1.15 JPCP 0.25 HMA-A 0.70 AS			1.20 JPCP 0.35 LCB 0.70 AS	1.25 JPCP 0.25 HMA-A 0.70 AS		
15.5 to 16	1.15 JPCP 0.35 LCB 0.70 AS	1.20 JPCP 0.25 HMA-A 0.70 AS			1.25 JPCP 0.35 LCB 0.70 AS	1.30 JPCP 0.23 HMA-A 0.70 AS		
16.5 to 17	1.20 JPCP 0.35 LCB 0.70 AS	1.25 JPCP 0.25 HMA-A 0.70 AS			1.30 JPCP 0.35 LCB 0.70 AS	1.35 JPCP 0.25 HMA-A 0.70 AS		
> 17	1.25 JPCP 0.35 LCB 0.70 AS	1.25 JPCP 0.25 HMA-A 0.70 AS			1.35 JPCP 0.35 LCB 0.70 AS	1.35 JPCP 0.25 HMA-A 0.70 AS		

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.15 ft sacrificial wearing course for future grinding of JPCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
AS = Class 2 Aggregate Subbase
TI = Traffic Index

Topic 624 – Engineering Procedures for Pavement Preservation

624.1 Preventive Maintenance

Examples of rigid pavement preventive maintenance strategies include the following or combinations of the following:

- Seal random cracks.
- Joint seal, repair/replace existing joint seals.
- Spall repair.
- Grooving.
- Grinding to restore surface texture.
- Special surface treatments (such as methacrylate, polyester concrete, and others). These strategies are normally used on bridge decks but can be applied, in limited situations, to rigid pavements for repair of problem areas.

Rigid pavement preventive maintenance strategies are discussed further in the Maintenance Manual, Chapter B.

624.2 Capital Preventive Maintenance (CAPM)

CAPM strategies include the following or combinations of the following:

(a) Slab replacement. The use of rapid strength concrete in the replacement of concrete slabs should be given consideration to minimize traffic impacts and open the facility to traffic in a minimal amount of time. Slab replacements may include replacing existing cement treated base or lean concrete base with rapid strength concrete. For further information (including information on rapid strength concrete) see the "Slab Replacement Guidelines" on the Department Pavement website.

(b) Grinding to correct faulting.

(c) Dowel bar retrofit. Guidelines for selecting and engineering dowel bar retrofit projects can be found on the Department Pavement website.

The roadway rehabilitation requirements for overlays (see Index 625.1(2)) and preparation of existing pavement surface (Index 625.1(3)) apply to CAPM projects. Additional details and information regarding CAPM policies and strategies can be found in Design Information Bulletin 81 "Capital Preventive Maintenance Guidelines" as well as the "Rigid Pavement CAPM and Rehabilitation Guidelines for Designers." Both can be found on the Department Pavement website.

Topic 625 - Engineering Procedures for Pavement and Roadway Rehabilitation

625.1 Rigid Pavement Rehabilitation Strategies

(1) Strategies. An overview of rigid pavement strategies for roadway rehabilitation is discussed in the "Rigid Pavement CAPM and Rehabilitation Guidelines for Designers," which can be found on the Department Pavement website. Some rehabilitation strategies discussed in the guide include the following or combinations of the following:

(a) Lane replacement. Lane replacements are engineered using the catalogs found in Index 623.1. Attention should be given to maintaining existing drainage patterns underneath the surface layer, (see Chapter 650 for further guidance). For further information see "Design Tools for Slab and Lane Replacements," on the Department Pavement website.

(b) Unbonded rigid overlay with flexible interlayer. To determine the thickness of the rigid layer, use the rigid layer thicknesses for new pavement found in Index 623.1. Include a 0.10 foot minimum flexible interlayer between the existing pavement and rigid overlay. The interlayer may need to be thicker if it is used temporarily for traffic handling.

(c) Crack, seat, and flexible overlay. The minimum standard thicknesses for a 20-year design life using this strategy are found in Table 625.1.

Table 625.1 is for a 20-year pavement design life. There are currently no standard crack, seat, and flexible overlay designs for pavement design lives greater than 20 years. For projects with longer than 20-year pavement design life, consider lane replacement, unbonded overlays, or consult Headquarters Office of Concrete Pavement and Pavement Foundations for possible experimental designs.

For crack, seat, and asphalt overlay projects, a nonstructural wearing course (such as an open graded friction course) may be placed in addition to (but not as a substitute for) the thickness found in Table 625.1. Once a rigid pavement has been cracked, seated, and overlaid with asphalt pavement it is considered to be a composite pavement and subsequent preservation and rehabilitation strategies are determined in accordance with the guidelines found in Chapter 640.

(d) Flexible overlay (without crack and seat). If the existing rigid pavement (JPCP) will not be cracked and seated, for a 20-year design life, add an additional 0.10 foot HMA to the minimum standard thicknesses of HMA surface course layer given in Table 625.1. Since the maximum thickness for RHMA-G is 0.20 foot (see Index 631.3), no additional thickness is needed if RHMA-G is used for the overlay.

(2) *Overlay Limits*. **On overlay projects, the entire traveled way and paved shoulder shall be overlaid.** Not only does this help provide a smoother finished surface, it also benefits bicyclists and pedestrians when they need to use the shoulder.

(3) *Preparation of Existing Pavement.* <u>Existing pavement distresses should be repaired before overlaying the pavement.</u> Cracks wider than ¼ inch should be sealed; loose pavement removed and patched; spalls repaired; and broken slabs or punchouts replaced. Existing thermoplastic traffic striping and above grade pavement markers should be removed. This applies to both lanes and adjacent shoulders (flexible and rigid). The Materials Report should include a reminder of these preparations. Crack sealants should be placed ¼ inch below grade to allow for expansion (i.e., recess fill) and to alleviate a potential bump if an overlay is placed. For information and criteria for slab replacements, see Chapter 2 of the Slab Replacement Guidelines on the Department Pavement website.

(4) *Selection.* The selection of the appropriate strategy should be based upon life-cycle costs, load transfer efficiency of the joints, materials testing, ride quality, safety, maintainability, constructibility, visual inspection of pavement distress, and other factors listed in Chapter 610. The Materials Report should discuss any historical problems observed in the performance of rigid pavement constructed with aggregates found near the proposed project and subjected to similar physical and environmental conditions.

625.2 Mechanistic-Empirical Method

For information on Mechanistic-Empirical Design application and requirements, see Index 606.3.

Topic 626 - Other Considerations

626.1 Traveled Way

(1) *Mainline.* No additional considerations.

(2) *Ramps and Connectors.* If tied rigid shoulders or widened slabs are used on the mainline, then the ramp or connector gore area (including ramp traveled way adjacent to the gore area) should also be constructed with rigid pavement (see Figure 626.1). This will minimize deterioration of the joint between flexible and rigid pavement. When the ramp or connector traveled way is rigid pavement, utilize the same base and thickness for the gore area as that to be used under the ramp traveled way, especially when concrete shoulders are utilized on the mainline. Note that in order to optimize constructability, any concrete pavement structure used for mainline concrete shoulders should still be perpetuated through the gore area. If the base is Treated Permeable Base (TPB) under the ramp's

Table 625.1

Minimum Standard Thicknesses for Crack, Seat, and Flexible Overlay[1]

TI <12.0	0.35′ HMA GPI or SAMI-R 0.10′ HMA (LC)	0.35′ HMA SAMI-F or SAMI-R 0.10′ HMA (LC)	0.20′ RHMA-G SAMI-R 0.10′ HMA (LC)
TI ≥12.0	0.40′ HMA GPI or SAMI-R 0.15′ HMA (LC)	0.20′ RHMA-G SAMI-R 0.15′ HMA (LC)	0.20′ RHMA-G 0.15′ HMA SAMI-F or SAMI-R 0.10′ HMA (LC)

NOTE:

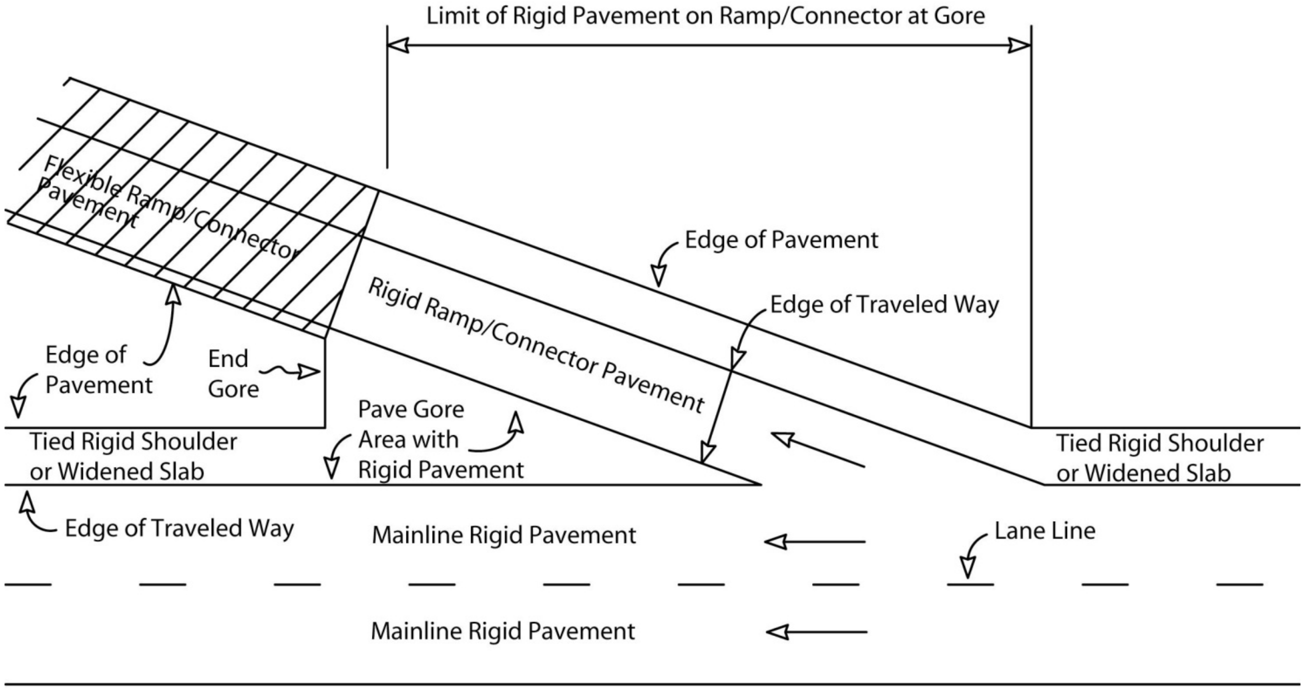
(1) If the existing rigid pavement is not cracked and seated, add minimum of 0.10 foot HMA above the SAMI layer.

Legend:

HMA = Hot Mix Asphalt
HMA (LC) = Hot Mix Asphalt Leveling Course
RHMA-G = Rubberized Hot Mix Asphalt (Gap Graded)
GPI = Geosynthetic Pavement Interlayer
SAMI-R = Stress Absorbing Membrane Interlayer (Rubberized)

Figure 626.1

Rigid Pavement at Ramp or Connector Gore Area



Notes: 1) Not all details shown
2) Off ramp shown. Same conditons apply for on ramps.

traveled way and shoulder, TPB should still be utilized in the ramp gore areas as well.

(3) *Ramp Termini.* Rigid pavement is sometimes placed at ramp termini instead of flexible pavement where there is projected heavy truck traffic (as defined in Index 613.5(1)(c)) topreclude pavement failure such as rutting or shoving from vehicular braking, turning movements, and oil dripping from vehicles. Once a design TI is selected for the ramp in accordance with Index 613.5, follow the requirements in Index 623.1 to engineer the rigid pavement structure for the ramp termini. The length of rigid pavement to be placed at the termini will depend on the geometric alignment of the ramp, ramp grades, and the length of queues of stopped traffic. The rigid pavement should extend to the first set of signal loops on signalized intersections. A length of 150 feet should be considered the minimum on unsignalized intersections. Special care should be taken to assure skid resistance in conformance with current standard specifications in the braking area, especially where oil drippage is concentrated. End anchors or transitions should be used at flexible/rigid pavement joints. The Department Pavement website has additional information and training for engineering pavement for intersections and rigid ramp termini.

626.2 Shoulder

The types of shoulders that are used for rigid pavements are shown in Figure 626.2A and can be categorized into the following three types:

(1) *Tied Rigid Shoulders.* These are shoulders that are built with rigid pavement that are tied to the adjacent lane with tie bars. These shoulders provide lateral support to the adjacent lane, which improves the long-term performance of the adjacent lane, reducing the need for maintenance or repair of the lane. To obtain the maximum benefit, these shoulders should be built monolithically with the adjacent lane (i.e., no contact joints). This will create aggregate interlock between the lane and shoulder, which provides increased lateral support. In order to build the lane and shoulder integrally, the shoulder cross slope needs to match the lane cross slope which may require a design exception (see Index 302.2 for further discussion).

The pavement structure for the tied rigid shoulder should match the pavement structure of the adjacent traffic lane. Special delineation of concrete shoulders may be required to deter the use of the shoulder as a traveled lane. District Traffic Operations should be consulted to determine the potential need for anything more than the standard edge stripe.

Tied rigid shoulders are the most adaptable to future widening and conversion to a lane. They should be the preferred shoulder type when future widening is planned within the design life of the pavement or where the shoulder will be used temporarily as a truck or bus lane. Where the shoulder is expected to be converted into a traffic lane in the future, the shoulder should be built to the same geometric and pavement standards as the lane. Additionally, the shoulder width should match the width of the future lane.

(2) *Widened Slab.* Widened slabs involve constructing the concrete panel for the lane adjacent to the shoulder 14 feet wide in lieu of the prescribed lane width. The additional width becomes part of the shoulder width and provides lateral support to the adjacent lane. Widened slabs provide as good or better lateral support than tied rigid shoulders at a lower initial cost provided that trucks and buses are kept at least 2 feet from the edge of the slab. A rumble strip or a raised pavement marking next to the pavement edge line of widened concrete slabs helps discourage trucks and buses from driving on the outside 2 feet of the slab. The use of rumble strips or raised markings requires approval from District Traffic Operations.

Widened slabs are most useful in areas where lateral support is desired but future widening is not anticipated or where there is a need to have a different cross slope on the shoulder than that of the adjacent lane.

(3) *Untied Shoulders.* Untied shoulders are flexible shoulders that are not built with a

widened slab or rigid shoulders that are not tied to the adjacent lane and not built adjacent to a widened slab. These shoulders do not provide lateral support to the adjacent lane. Although non-supported shoulders may have lower initial costs, they do not perform as well as tied rigid shoulders or widened slabs, which can lead to higher maintenance costs, user delays, and life cycle costs.

(4) *Selection Criteria.* It is preferred that shoulders be constructed of the same material as the traveled way pavement (in order to facilitate construction, improve pavement performance, and reduce maintenance cost). However, shoulders adjacent to rigid pavement traffic lanes can be either rigid or flexible with the following conditions:

(a) Tied rigid shoulders shall be used for:

- **Rigid pavements constructed in the High Mountain and High Desert climate regions (see climate map in Topic 615).**
- **Paved buffers between rigid High-Occupancy Vehicle (HOV) lanes and rigid mixed flow lanes. Same for High-Occupancy Toll (HOT) lanes.**
- **Rigid ramps to and from truck inspection stations.**

(b) Either tied rigid shoulders or widened slabs shall be used for:

- **Continuously reinforced concrete pavement.**
- **Horizontal radii 300 feet or less.**
- **Truck and bus only lanes.**

Where tied rigid shoulders or widened slabs are used, they shall continue through ramp and gore areas (see Figure 626.2B).

Because heavy trucks cause deterioration by repeated heavy loading on the outside edge of pavement, at the corners, and the midpoint of the slab, widened slabs or tied rigid shoulders should be used for heavy truck routes with a TI greater than or equal to 14.0.

In those instances where flexible shoulders are used with rigid pavement, the minimum flexible shoulder thickness should be determined in accordance with Topic 633.

These conditions apply to all rigid pavement projects including new construction, reconstruction, widening, adjacent lane replacements, and shoulder replacements. Typically existing flexible shoulders next to rigid pavement are not replaced for rehabilitation projects that involve only grinding, dowel bar retrofits, and individual slab replacements. Consideration should be given to replacing flexible shoulders with tied rigid shoulders or widened slabs when the adjacent lane is being replaced or overlaid with a rigid pavement. The District determines when an existing flexible shoulder is replaced with a rigid shoulder or widened slab.

The shoulder pavement structure selected must meet or exceed the pavement design life standards in Topic 612. In selecting whether to construct rigid or flexible shoulders the following factors should be considered:

- Life-cycle cost of the shoulder.
- Ability and safety of maintenance crews to maintain the shoulder. In confined areas, such as in front of retaining walls or narrow shoulders, and on high volume roadways (AADT > 150,000) consideration should be given to engineering a shoulder that requires the least amount of maintenance, even if it is more expensive to construct.
- Future plans to widen the facility or convert the shoulder to a traffic lane.
- Width of shoulder. When shoulder widths are less than 5 feet, tied rigid shoulders are preferable to a widened rigid slab and narrow flexible shoulder, less than 3 feet, for both constructibility and maintainability.
- For projects where the tracking width lines are shown to encroach onto paved shoulders or any portion of the gutter pan, tied rigid shoulders and the gutter pan

structure must be engineered to sustain the weight of the design vehicle. See Topic 404 for design vehicle guidance.

See Index 1003.5(1)) for surface quality guidance for highways open to bicyclists.

626.3 Intersections

Standard joint spacing patterns found in the Standard Plans do not apply to intersections. Special paving details for intersections need to be included in the project plans. Special consideration needs to be given to the following features when engineering a rigid pavement intersection:

- Intersection limits.
- Joint types and joint spacing.
- Joint patterns.
- Slab dimensions.
- Pavement joints at utilities.
- Dowel bar and tie bar placement.

Additional information and training is available on the Department Pavement website.

626.4 Roadside Facilities

(1) *Safety Roadside Rest Areas and Vista Points.* If rigid pavement is selected for some site-specific reason(s), the pavement structures used should be sufficient to handle projected loads at most roadside facilities. To select the pavement structure, determine the Traffic Index either from traffic studies and projections developed for the project or the values found in Table 613.5B, whichever is greater. Then select the appropriate pavement structure from the catalog in Index 623.1.

Joint spacing patterns found in the Standard Plans do not apply to parking areas. Joint patterns should be engineered as square as possible. Relative slab dimensions should be approximately 1:1 to 1:1.25, transverse-to-longitudinal. Transverse and longitudinal joints should be perpendicular to each other. Joints are doweled in one direction and tied in the other in accordance with Index 622.4. Special attention should be given to joint patterns around utility covers and manholes.

Use guidelines for intersections in Index 626.3 for further information.

(2) *Park and Ride Facilities.* Flexible pavement should be used for park and ride facilities. If transit buses access the park and ride facility, use the procedures for bus pads in this Index for engineering bus access.

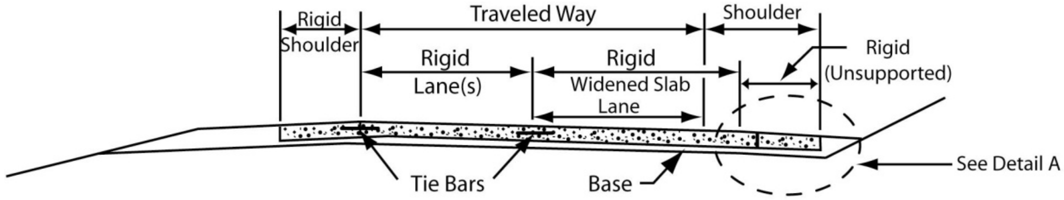
(3) *Bus Pads.* Bus pads are subjected to similar stresses as intersections; however, it is not practical to engineer rigid bus pads according to the Traffic Index, or according to bus counts. The minimum pavement structure for bus pads should be 0.85 foot JPCP with dowel bars at transverse joints on top of 0.5 foot lean concrete base or Type A hot mix asphalt (0.75 foot CRCP may be substituted for 0.85 foot JPCP). For Type II soil as described in Table 623.1A, include 0.5 foot of aggregate subbase. Type III soil should be treated in accordance with Index 614.4. Where local standards are more conservative than the pavement structures mentioned above, local standards should govern.

Relative slab dimensions for bus pads should be approximately 1:1 to 1:1.25, transverse-to-longitudinal. The width of the bus pad should be no less than the width of the bus plus 4 feet. If the bus pad extends into the traveled way, the rigid bus pad should extend for the full width of the lane occupied by buses. The minimum length of the bus pad should be 1.5 times the length of the bus(es) that will use the pad at any given time. This will provide some leeway for variations in where the bus stops. Additional length of rigid pavement should be considered for approaches and departures from the bus pad since these locations may be subjected to the same stresses from buses as the pad. A 115-foot length of bus pad (which is approximately 250 percent to 300 percent times the length of typical 40-foot buses) should provide sufficient length for bus approach and departure. The decision whether to use rigid pavement for bus approach and departure to/from bus pads is the responsibility of the District.

An end anchor may improve long-term performance at the flexible-to-rigid pavement transition. Doweled transverse joints should

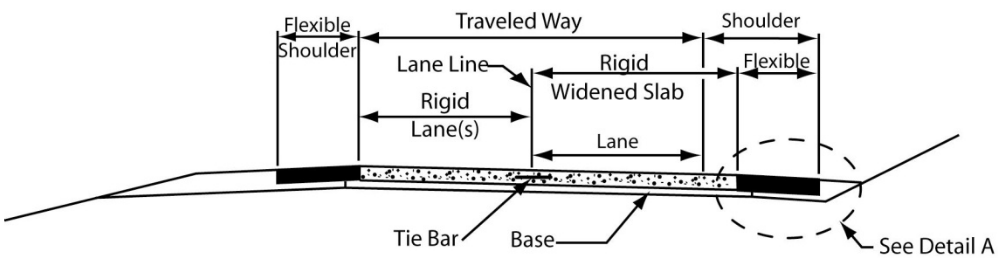
Figure 626.2A

Rigid Pavement and Shoulder Details



RIGID SHOULDERS

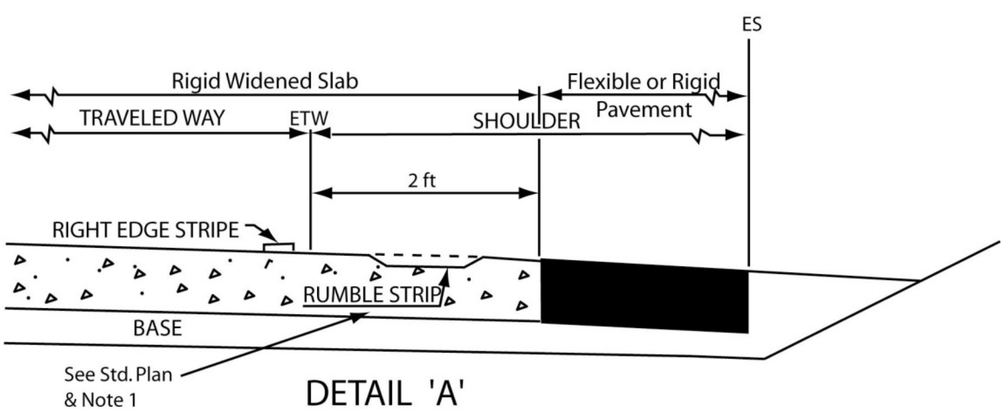
No Scale



FLEXIBLE SHOULDERS

No Scale

NOTE: These illustrations are only to show nomenclature and are not to be used for geometric cross section details.



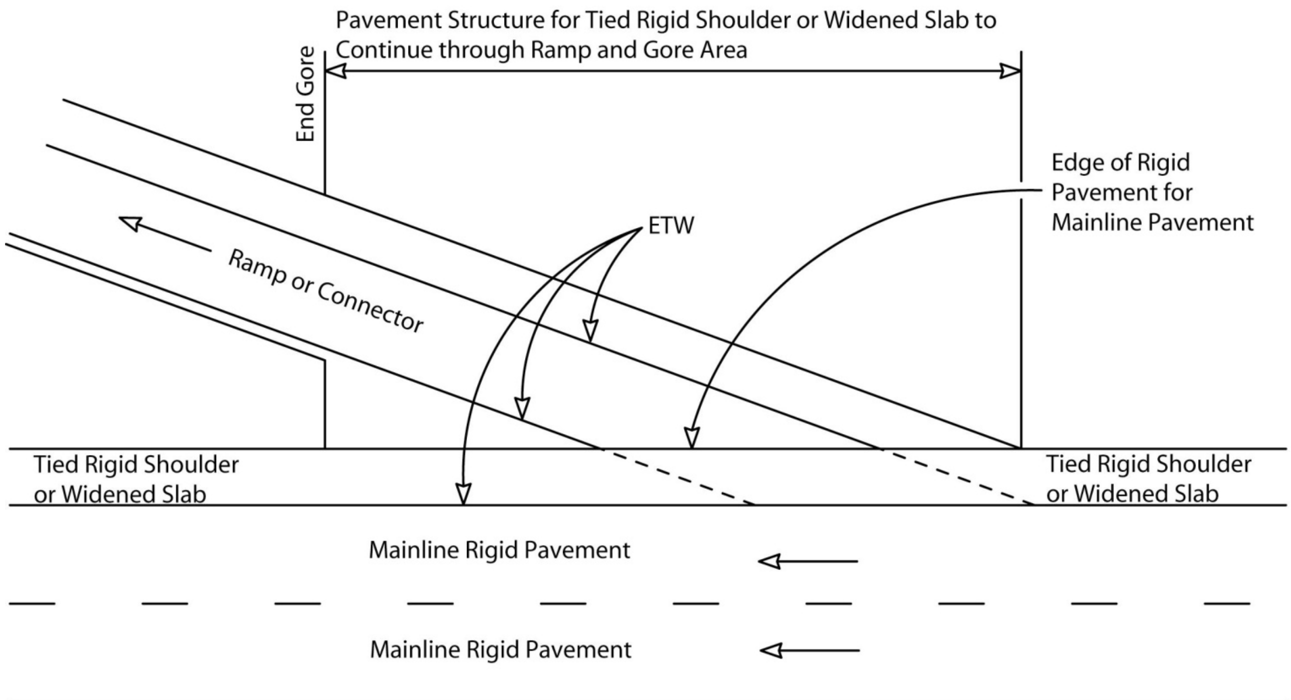
DETAIL 'A'

NOTES: 1. Use of Rumble Strips is determined in consultation with District Traffic Operations.

2. Right side widened slab is shown. Left side widened slab is similar.

Figure 626.2B

Rigid Shoulders Through Ramp and Gore Areas

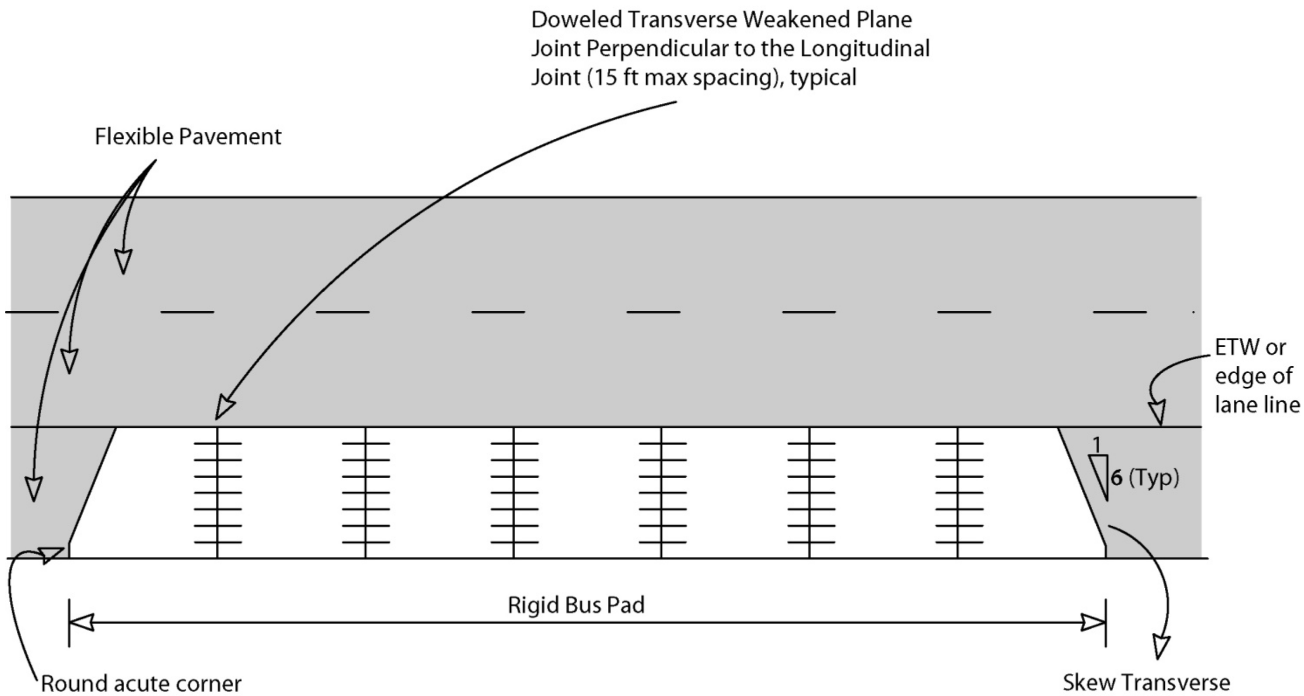


Notes: 1) Not all details shown
2) Off ramp shown. Same conditions apply for on ramps.

be perpendicular to the longitudinal joint at maximum 15 feet spacing, but consider skewing (at 1:6 typical) entrance/exit transverse flexible-to-rigid transitions, note that since acute corners can fail prematurely, acute corners should be rounded (see Figure 626.4). Special care should be taken to assure skid resistance in conformance with current Standard Specifications in the braking area, especially where oil drippage is concentrated.

Figure 626.4

Rigid Bus Pad



Note: Not all details shown

CHAPTER 630 FLEXIBLE PAVEMENT

Topic 631 - Types of Flexible Pavements & Materials

Index 631.1 - Hot Mix Asphalt (HMA)

HMA consists of a mixture of asphalt binder and a graded aggregate ranging from coarse to very fine particles. The aggregate can be treated and the binder can be modified. HMA could be made from new or recycled material. Examples of recycled asphalt include, but are not limited to, hot and cold in-place recycling. HMA is classified by type depending on the specified aggregate quality and mix design criteria appropriate for the project conditions. HMA types are found in the Standard Specifications and Standard Special Provisions.

631.2 Open Graded Friction Course (OGFC)

OGFC (formerly known as open graded asphalt concrete (OGAC)) is a non-structural wearing course used primarily on HMA. It is occasionally used with modified binders on rigid pavements. The primary benefit of using OGFC is the improvement of wet weather skid resistance, reduced potential for hydroplaning, reduced water splash and spray, and reduced night time wet pavement glare. Secondary benefits include better wet-night visibility of traffic lane stripes and pavement markers, and better wet weather (day and night) delineation between the traveled way and shoulders.

For information and applicability of OGFC in new construction and rehabilitation projects refer to OGFC Guideline available on the Department Pavement website. Also, see Maintenance Technical Advisory Guide (MTAG) for additional information and use of OGFC in pavement preservation.

631.3 Rubberized Hot Mix Asphalt (RHMA)

Rubberized asphalt is formulated by mixing granulated (crumb) rubber with hot asphalt to form an elastic binder with less susceptibility to temperature changes. The rubberized asphalt is substituted for the regular asphalt as the binder for the flexible pavement. This is called the wet method. Other methods of using rubber in flexible pavements are available. See Asphalt Rubber Usages Guide (ARUG), available on the Department Pavement website, for further details.

RHMA is generally specified to retard reflection cracking, resist thermal stresses created by wide temperature variations and add flexibility to a structural overlay. At present, the Department uses gap-graded (RHMA-G) and open-graded (RHMA-O) rubberized asphalt. The difference between the two is in the gradation of the aggregate. RHMA-O is used only as a non-structural wearing course. RHMA-G can be used as either a surface course or a non-structural wearing course. RHMA should be considered the strategy of choice when evaluating alternatives for a project. If RHMA is found to be inappropriate due to availability, constructibility, environmental factors, or cost, it shall be documented in the scoping document, Project Initiation Document (PID), or Project Report (PR).

The minimum thickness for RHMA (any type) should be 0.10 foot for new construction and rehabilitation. For pavement preservation, RHMA may be placed as thin as 0.08 foot provided compaction requirements can be met. The maximum thickness for RHMA-G is 0.20 foot. The maximum thickness for RHMA-O is 0.15 foot. If a thicker surface layer or overlay is called for, then a HMA layer should be placed prior to placing the RHMA. RHMA should only be placed over a flexible or rigid surface course and not on a granular layer. RHMA-O may be placed on top of new RHMA-G. Do not place conventional HMA or OGFC over new RHMA pavement.

It is undesirable to place RHMA-G or RHMA-O in areas that will not allow surface water to drain. As an example, a surface that is milled only on the traveled way and not on the shoulder forms a "bathtub" section that can trap water beneath the surface of the traveled way. To prevent this effect, RHMA-G should be placed over the whole cross section of the road (traveled way and shoulders).

For additional information and applicability of RHMA in new construction and rehabilitation projects refer to Asphalt Rubber Usage Guide available on the Department Pavement website.

631.4 Other Types of Flexible Pavement

There are other types of flexible pavements such as cold mix, Resin Pavement, and Sulphur Extended Hot Mix Asphalt. The other types of pavements are either used for maintenance treatments or not currently used on State highways. For pavement preservation and other maintenance treatments refer to the Department's Maintenance Manual.

631.5 Stress Absorbing Membrane Interlayers (SAMI)

SAMI are used with flexible layer rehabilitation as a means to retard reflective cracks, prevent water intrusion, and (in the case of SAMI-R (rubberized)) enhance pavement structural strength. Two types of SAMI are:

- Rubberized (SAMI-R). SAMI-R is a rubberized chip seal.
- Geosynthetic Pavement Interlayer (GPI), consists of asphalt-imbued geotextile.

Sound engineering judgment is required when considering the use of a SAMI.

- Consideration should be given to areas that may prohibit surface water from draining out the sides of the overlay, thus forming a "bathtub" section.
- Since SAMI-R can act as a moisture barrier, it should be used with caution in hot environments where it could prevent underlying moisture from evaporating.
- When placed on an existing pavement, preparation is required to prevent excess stress on the membrane. This includes sealing cracks wider than ¼ inch and repairing potholes and localized failures.

A SAMI may be placed between layers of new flexible pavement, such as on a leveling course, or on the surface of an existing flexible pavement. A GPI should not be placed directly on coarse surfaces such as a chip seal, OGFC, areas of numerous rough patches, or on a pavement that has been cold planed. Coarse surfaces may penetrate the fabric and the paving asphalt binder used to saturate the fabric may collect in the voids or valleys leaving areas of the fabric dry. For the GPI to be effective in these areas, use a layer of HMA prior to the placement of the GPI.

GPI is ineffective in the following applications:

- When placed under rubberized hot mix asphalt (RHMA). This is due to the high placement temperature of the RHMA-G mix, which is close to the melting temperature of the GPI.
- For providing added structural strength when placed in combination with new flexible pavement.
- In the reduction of thermal cracking of the new flexible pavement overlay.

Topic 632 - Engineering Criteria

632.1 Engineering Properties

(1) *Smoothness.* The smoothness of a pavement impacts its ride quality, overall durability, and performance. Ride quality (which is measured by the smoothness of ride) is also the highest concern listed in public surveys on pavement condition. Smoothness specifications have been improved and incentive/disincentive specifications have been developed to assure designed smoothness values are achieved in construction. Incentive / disincentive specifications can be used where the project meets the warrants for the specification. For up to date and additional information on smoothness and the application of the smoothness specifications see the smoothness page on the Department Pavement website.

(2) *Asphalt Binder Type.* Asphalt binders are most commonly characterized by their physical properties. An asphalt binder's physical properties directly relate to field

performance. Although asphalt binder viscosity grading is still common, new binder tests and specifications have been developed to more accurately characterize temperature extremes which pavements in the field are expected to withstand. These tests and specifications are specifically designed to address three specific pavement distress modes: permanent deformation (rutting), fatigue cracking, and low temperature cracking.

In the past, the Department has classified unmodified asphalt binder using viscosity grading based on the Aged Residue (AR) System and Performance Based Asphalt (PBA) binder system. Beginning January 1, 2006, the Department switched to the nationally recognized Performance Grade (PG) System for conventional binders. Effective from January 1, 2007, the Department has graded polymer-modified binders as Performance Graded-Polymer Modified (PG-PM) binder in lieu of PBA.

Performance grading is based on the concept that asphalt binder properties should be related to the conditions under which the binder is used. PG asphalt binders are selected to meet expected climatic conditions as well as traffic speed and volume adjustments. Therefore, the PG system uses a common set of tests to measure physical properties of the binder that can be directly related to field performance of the pavement at its service temperatures. For example, a binder identified as PG 64–10 must meet performance criteria at an average seven-day maximum pavement temperature of 64°C and also at a minimum pavement temperature of –10°C.

Although modified asphalt binder is more expensive than unmodified binder, in hot mix asphalt (HMA), it can provide improved performance and durability for sensitive climate conditions. While unmodified binder is adequate for most applications, improved resistance to rutting, thermal cracking, fatigue damage, stripping, and temperature susceptibility have led polymer modified binders to be substituted for conventional asphalt in many paving and maintenance applications.

Table 632.1 provides the binder grade that is to be used for each climatic region for general application. For HMA, values are given for typical and special conditions. For a few select applications such as dikes and tack coats, PG binder requirements are found in the applicable Standard Specifications or Standard Special Provisions.

For locations of each pavement climate region see Topic 615.

Special conditions are defined as those roadways or portion of roadways that need additional attention due to conditions such as:

- Heavy truck/bus traffic (over 10 million ESALs for 20 years).
- Truck/bus stopping areas (parking area, rest area, loading area, etc.).
- Truck/bus stop and go areas (intersections, metered ramps, ramps to and from Truck Scales etc.).
- Truck/bus climbing and descending lanes.

The final decision as to whether a roadway meets the criteria for special conditions rests with the District. It should be noted that even though special binder grades help meet the flexible pavement requirements for high truck/bus use areas, they should not be considered as the only measure needed to meet these special conditions. The District Materials Engineer should be consulted for additional recommendations for these locations.

For more detailed information on PG binder selection, refer to the Department Pavement website.

632.2 Performance Factors

The procedures and practices found in this chapter are based on research and field experimentation undertaken by the Department and AASHTO. These procedures were calibrated for pavement design lives of 10 to 20 years and Traffic Index (TI) ranging from 5.0 to 12. Extrapolations and supplemental requirements were subsequently

Table 632.1

Asphalt Binder Grade

Binder / Climatic Region	Binder Grades for Hot Mixed Asphalt (HMA)[1,2]				
	Dense Graded HMA		Open Graded HMA		Gap and Open Graded Rubberized Hot Mix Asphalt (RHMA)
			Placement Temperature		
	Typical	Special[3]	> 70°F	≤ 70°F	
South Coast Central Coast Inland Valley	PG 64-10	PG 70-10 or PG 64-28 PM	PG 64-10	PG 58-34 PM	PG 64-16
North Coast	PG 64-16	PG 64-28 PM	PG 64-16	PG 58-34 PM	PG 64-16
Low Mountain South Mountain	PG 64-16	PG 64-28 PM	PG 64-16	PG 58-34 PM	PG 64-16
High Mountain High Desert	PG 64-28	PG 58-34 PM[4]	PG 64-28	PG 58-34 PM	PG 58-22
Desert	PG 70-10	PG 64-28 PM	PG 70-10	PG 58-34 PM or PG 64-28 PM[3]	PG 64-16

NOTES:

(1) PG = Performance Graded

(2) PM = Polmer Modified

(3) PG 76-22 PM may be specified for conventional dense graded hot mix asphalt for special conditions in all climatic regions when specifically requested by the District Materials Engineer.

(4) PG 64-28 may be specified when specifically requested by the District Materials Engineer.

(5) Consult the District Materials Engineer for which binder grade to use.

developed to address longer pavement design lives and higher Traffic Indices. Details on mix design and other requirements for these procedures are provided in the Standard Specifications and Standard Special Provisions. Alterations to the requirements in these documents can impact the performance of the pavement structure and the performance values found in this chapter.

Topic 633 - Engineering Procedures for New and Reconstruction Projects

633.1 Empirical Method

The data needed to engineer a flexible pavement are California R-value of the subgrade and the TI for the pavement design life. Engineering of the flexible pavement is based on a relationship between the gravel equivalent (GE) of the pavement structural materials, the TI, and the California R-value of the underlying material. The relationship was developed by the Department through research and field experimentation.

The procedures and rules governing flexible pavement engineering are as follows, (Sample calculations are provided on the Department Pavement website.):

(1) *Procedures for Engineering Multiple Layered Flexible Pavement.*

The California Department of Transportation empirical method, commonly referred to as the Hveem method, for determining design thicknesses of the structural layers of flexible pavement structure involves the determination of the following design parameters:

- Traffic Index (TI)
- California R-value (R)
- Gravel Equivalent (GE), and
- Gravel Factor (G_f)

Once TI, R, GE, and G_f are determined, then the design thickness of each structural layer is determined using the Hveem method. These design parameters and the Hveem design method are discussed in the following sections:

(a) As discussed in Index 613.3(3), the TI is a measure of the cumulative number of ESALs expected during the design life of the pavement structure. The TI is determined to the nearest 0.5 using the equation given in Index 613.3(3) or from Table 613.3C.

(b) The California R-value is a measure of resistance of soils to deformation under wheel loading and saturated soils conditions. The California R-value is determined as discussed in Index 614.3.

(c) The gravel equivalent (GE) of each layer or the entire flexible pavement structure is the thickness of gravel (aggregate subbase) that would be required to prevent permanent deformation in the underlying layer or layers due to cumulative traffic loads anticipated during the design life of the pavement structure. The GE requirement of the entire flexible pavement or each layer is calculated using the following equation:

$$GE = 0.0032(TI)(100 - R)$$

Where:

GE = Gravel Equivalent in feet

TI = Traffic Index

R = California R-value of the material below the layer or layers for which the GE is being calculated.

The GE requirement of each type of material used in the flexible pavement structure is determined for each structural layer, starting with the surface course and proceeding downward to base and subbase as needed. For pavements that include base and/or subbase, a safety factor of 0.20 foot is added to the GE requirement for the surface course to compensate for construction tolerances allowed by the contract specifications. Since the safety factor is not intended to increase the GE of the overall pavement, a compensating thickness is subtracted from the subbase layer (or base layer if there is no subbase). For pavements that are full depth asphalt, a safety factor of 0.10 foot is added to the required GE of the pavement structure. When determining the appropriate safety factor to be added, Hot Mix Asphalt Base (HMAB) and Asphalt Treated Permeable Base (ATPB) should be considered as part of the surface course.

(d) The gravel factor (G_f) of pavement structural material is the relative strength of that material compared to gravel. Gravel factors for HMA decrease as TI increases, and also increase with HMA thickness greater than 0.5 foot; while G_f for base and subbase materials are only dependent on the material type.

The G_f of HMA varies with layer thickness (t) for any given TI as follows:

$t \leq 0.50$ ft:	$G_f = \frac{5.67}{(TI)^{1/2}}$
$t > 0.50$ ft:	$G_f = (7.00)\frac{(t)^{1/3}}{(TI)^{1/2}}$

These equations are valid for TIs ranging from 5 to 15. For TIs greater than 15, use a rigid or composite pavement or contact the Headquarters Division of Maintenance – Pavement Program for experimental options. For TIs less than 5, use a TI=5. Typical gravel factors for HMA of thickness equal to or less than 0.5 foot, and various types of base and subbase materials, are provided in table 633.1. Additional information on G_f for base and subbase materials are provided in Table 633.1B.

(e) The design thickness of each structural layer of flexible pavement is obtained either by dividing the GE by the appropriate gravel factor for that layer material, or from Table 633.1. The layer thickness determined by dividing GE by G_f is rounded up to the next higher value in 0.05-foot increments.

$$\text{Thickness (t)} = \frac{GE}{G_f}$$

The minimum thickness of any asphalt layer should not be less than twice the maximum aggregate size, and the minimum thickness of the surface course should not be less than 0.15 foot. The limit thicknesses for placing HMA for each TI, and the limit thickness for each type of base and subbase materials, are shown in Table 633.1

Base and subbase materials, other than ATPB, should each have a minimum thickness of 0.35 foot. When the calculated thickness of base or subbase material is less than the desired 0.35 foot minimum thickness, either: (a) increase the thickness to the minimum without changing the thickness of the overlying layers or (b) eliminate the layer and increase the thickness of the overlying layers to compensate for the reduction in GE.

Generally, the layer thickness of Lime Treated Subbase (LTS) should be limited, with 0.65 foot as the minimum and 2 feet as the maximum. A surface layer placed directly on the LTS should have a thickness of at least 0.25 foot.

The thicknesses determined by the procedures outlined in this section are not intended to preclude other combinations and thicknesses of materials. Adjustments to the thickness of the various materials may be made to accommodate construction restrictions or practices, and minimize costs, provided the minimum thicknesses, maximum thicknesses, and minimum GE requirements (including safety factors) of the entire pavement structure and each layer are as specified.

(2) *Procedures for Full Depth Hot Mix Asphalt.* Full depth hot mix asphalt applies when the pavement structure is comprised entirely of a flexible surface layer in lieu of base and subbase. The flexible surface layer may be comprised of a single or multiple types of

flexible pavements including HMA, RHMA, interlayers, special asphalt binders, or different mix designs. Considerations regarding worker safety, short construction windows, the amount of area to be paved, or temporary repairs may make it desirable in some instances to reduce the total thickness of the pavement by placing full depth hot mix asphalt. Full depth hot mix asphalt also is less affected by moisture or frost, does not let moisture build up in the subgrade, provides no permeable layers that entrap water, and is a more uniform pavement structure. Use the standard equation in Index 633.1(1) with the California R-value of the subgrade to calculate the initial GE for the entire pavement structure. Increase this by adding the safety factor of 0.10 foot to obtain the required GE for the flexible pavement. Then refer to Table 633.1, select the closest layer thickness for conventional hot mixed asphalt, and determine the adjusted GE that it provides. The GE of the safety factor is not removed in this design. Adjust the final thickness as needed when using other types of materials than hot mixed asphalt.

A Treated Permeable Base (TPB) layer may be placed below full depth hot mix asphalt on widening projects to perpetuate, or match, an existing treated permeable base layer for continuity of drainage. Reduce the GE of the surface layer by the amount of GE provided by the TPB. In no case should the initial GE of the surface layer over the TPB be less than 40 percent of the GE required over the subbase as calculated by the standard engineering equation. When there is no subbase, use 50 for the California R-value for this calculation. In cases where a working table will be used, the GE of the working table is subtracted from the GE of the surface layer as well. A working table is a minimum thickness of material, asphalt, cement, or granular based, used to place construction equipment and achieve compaction requirements when compaction is difficult or impossible to meet.

(3) *Modifications for Pavement Design Life Greater than 20 Years.* The above procedure is based on an empirical method for a twenty-year pavement design life. For pavement design lives greater than twenty years, in addition to using a TI for that longer design life, provisions should be made to increase material durability and other appropriate measures to protect pavement layers from degradation.

The following enhancements shall be incorporated into all flexible pavements with a design life greater than twenty years:

- Use the procedures for full depth hot mix asphalt to determine the minimum thickness for flexible pavement. Cement treated base or lean concrete base can be used in lieu of hot mix asphalt but not in lieu of aggregate base, aggregate subbase, or a treated permeable base.
- Place a minimum 0.50 foot of Class 2 Aggregate base underneath the flexible pavement. This aggregate base layer is not considered part of the pavement structural design and cannot be used to reduce the thickness of the full depth hot mix asphalt layer.
- Use a non-structural wearing course (such as OGFC) above the surface layer (minimum 0.10 foot). See Index 602.1(5) for further details.
- Use rubberized hot mix asphalt (maximum 0.20 foot) or a PG-PM binder (minimum 0.20 foot) for the top of the surface layer.

The following enhancements should be incorporated into all flexible pavements with a pavement design life greater than twenty years when recommended by the District Materials Engineer:

(a) Use higher asphalt binder content for bottom of the surface layer (rich-bottom concept) and using higher stiffness asphalt binder.

(b) Utilize subgrade enhancement fabrics at the subgrade for California R-values less than 40.

(c) Use SAMIs within the surface layer.

Table 633.1
Gravel Equivalents (GE) and Thickness of Structural Layers (ft)

Actual Layer Thickness (ft) [5]	HMA[1], [2]											Base and Subbase [3]					
	Traffic Index (TI)											TI is not a factor					
	5.0 & below	5.5 6.0	6.5 7.0	7.5 8.0	8.5 9.0	9.5 10.0	10.5 11.0	11.5 12.0	12.5 13.0	13.5 14.0	14.5 15.0	HMAB; LCB	CTPB; CTB (Cl. A)	ATPB	CTB (Cl. B)	AB	AS
	G_f (For HMA thickness equal to or less than 0.5 ft, G_f decreases with TI) [4]											G_f (Constant for any base or subbase material irrespective of TI or thickness)					
	2.54	2.32	2.14	2.01	1.89	1.79	1.71	1.64	1.57	1.52	1.46	1.9	1.7	1.4	1.2	1.1	1.0
	GE for HMA layer (ft)											GE for Base or Subbase layer (ft)					
0.10	0.25	0.23	0.21	0.20	0.19	0.18	0.17	0.16	0.16	0.15	0.15	--	--	--	--	--	--
0.15	0.38	0.35	0.32	0.30	0.28	0.27	0.26	0.25	0.24	0.23	0.22	--	--	--	--	--	--
0.20	0.51	0.46	0.43	0.40	0.38	0.36	0.34	0.33	0.31	0.30	0.29	--	--	--	--	--	--
0.25	0.63	0.58	0.54	0.50	0.47	0.45	0.43	0.41	0.39	0.38	0.37	--	--	0.35	--	--	--
0.30	0.76	0.69	0.64	0.60	0.57	0.54	0.51	0.49	0.47	0.45	0.44	--	--	0.42	--	--	--
0.35	0.89	0.81	0.75	0.70	0.66	0.63	0.60	0.57	0.55	0.53	0.51	0.67	0.60	0.49	0.42	0.39	0.35
0.40	1.01	0.93	0.86	0.80	0.76	0.72	0.68	0.65	0.63	0.61	0.59	0.76	0.68	0.56	0.48	0.44	0.40
0.45	1.14	1.04	0.96	0.90	0.85	0.81	0.77	0.74	0.71	0.68	0.66	0.86	0.77	0.63	0.54	0.50	0.45
0.50	1.27	1.16	1.07	1.00	0.94	0.90	0.85	0.82	0.79	0.76	0.73	0.95	0.85	0.70	0.60	0.55	0.50
0.55	1.41	1.29	1.19	1.12	1.05	1.00	0.95	0.91	0.87	0.84	0.81	1.05	0.94	0.77	0.66	0.61	0.55
0.60	1.58	1.45	1.34	1.25	1.18	1.12	1.07	1.02	0.98	0.95	0.91	1.14	1.02	0.84	0.72	0.66	0.60
0.65	1.76	1.61	1.49	1.39	1.31	1.25	1.19	1.14	1.09	1.05	1.02	1.24	1.11	0.91	0.78	0.72	0.65
0.70	--	1.78	1.64	1.54	1.45	1.38	1.31	1.26	1.21	1.16	1.12	1.33	1.19	--	0.84	0.77	0.70
0.75	--	1.95	1.80	1.69	1.59	1.51	1.44	1.38	1.32	1.27	1.23	1.43	1.28	--	0.90	0.83	0.75
0.80	--	2.12	1.96	1.84	1.73	1.64	1.57	1.50	1.44	1.39	1.34	1.52	1.36	--	0.96	0.88	0.80
0.85	--	--	2.13	1.99	1.88	1.78	1.70	1.63	1.56	1.51	1.46	1.62	1.45	--	1.02	0.94	0.85
0.90	--	--	2.30	2.15	2.03	1.92	1.83	1.76	1.69	1.63	1.57	1.71	1.53	--	1.08	0.99	0.90
0.95	--	--	--	2.31	2.18	2.07	1.97	1.89	1.81	1.75	1.69	1.81	1.62	--	1.14	1.05	0.95
1.00	--	--	--	2.47	2.33	2.21	2.11	2.02	1.94	1.87	1.81	1.90	1.70	--	1.20	1.10	1.00
1.05	--	--	--	2.64	2.49	2.36	2.25	2.16	2.07	2.00	1.93	2.00	1.79	--	1.26	1.16	1.05
1.10	--	--	--	--	2.65	2.51	2.40	2.29	2.20	2.12	2.05	--	--	--	--	--	1.10
1.15	--	--	--	--	2.81	2.67	2.54	2.43	2.34	2.25	2.18	--	--	--	--	--	1.15
1.20	--	--	--	--	2.98	2.82	2.69	2.58	2.48	2.39	2.30	--	--	--	--	--	1.20
1.25	--	--	--	--	--	2.98	2.84	2.72	2.61	2.52	2.43	--	--	--	--	--	1.25
1.30	--	--	--	--	--	3.14	2.99	2.87	2.75	2.65	2.56	--	--	--	--	--	1.30
1.35	--	--	--	--	--	3.30	3.15	3.01	2.90	2.79	2.70	--	--	--	--	--	--
1.40	--	--	--	--	--	--	3.31	3.16	3.04	2.93	2.83	--	--	--	--	--	--
1.45	--	--	--	--	--	--	3.46	3.32	3.19	3.07	2.97	--	--	--	--	--	--
1.50	--	--	--	--	--	--	3.62	3.47	3.33	3.21	3.10	--	--	--	--	--	--
1.55	--	--	--	--	--	--	--	3.62	3.48	3.36	3.24	--	--	--	--	--	--
1.60	--	--	--	--	--	--	--	3.78	3.63	3.50	3.38	--	--	--	--	--	--
1.65	--	--	--	--	--	--	--	3.94	3.79	3.65	3.52	--	--	--	--	--	--
1.70	--	--	--	--	--	--	--	--	3.94	3.80	3.67	--	--	--	--	--	--
1.75	--	--	--	--	--	--	--	--	4.09	3.95	3.81	--	--	--	--	--	--
1.80	--	--	--	--	--	--	--	--	4.25	4.10	3.96	--	--	--	--	--	--
1.85	--	--	--	--	--	--	--	--	--	4.25	4.10	--	--	--	--	--	--
1.90	--	--	--	--	--	--	--	--	--	4.40	4.25	--	--	--	--	--	--
1.95	--	--	--	--	--	--	--	--	--	4.56	4.40	--	--	--	--	--	--
2.00	--	--	--	--	--	--	--	--	--	--	4.55	--	--	--	--	--	--

Notes:

(1) Open Graded Friction Course (conventional and rubberized) is a non-structural wearing course and provides no structural value.

(2) Top portion of HMA surface layer (maximum 0.20 ft.) may be replaced with equivalent RHMA-G thickness. See Topic 631.3 for additional details.

(3) See Table 663.1B for additional information on Gravel Factors (G_f) and California R-values for base and subbase materials.

(4) These G_f values are for TIs shown and HMA thickness equal to or less than 0.5 foot only. For HMA thickness greater than 0.5 foot, appropriate G_f should be determined using the equation in Index 633.1(1)(c).

(5) For HMA layer, select TI range, then go down to the appropriate GE and across to the thickness column. For base and subbase layer, select material type, then go down to the appropriate GE and across to the thickness column.

(d) Use a separation fabric above granular layers. Note that the fabric used needs to be able to resist construction loads or construction equipment must be able to keep off of the fabric.

(4) *Alternate Procedures and Materials.* At times, experimental procedures and/or alternative materials are proposed as part of the design or construction. See Topic 606 for further discussion.

633.2 Mechanistic-Empirical Method

For information on Mechanistic-Empirical Design application and requirements, see Index 606.3.

Topic 634 - Engineering Procedures for Flexible Pavement Preservation

634.1 Preventive Maintenance

For details regarding preventive maintenance strategies for flexible pavement, see the "Maintenance Technical Advisory Guide" on the Department Pavement website. Deflection studies are not required for preventive maintenance projects.

634.2 Capital Preventive Maintenance (CAPM)

The standard design for a flexible pavement CAPM project with an International Roughness Index (IRI) less than 170 inches per mile at PS&E is an overlay of either 0.15 foot of rubberized hot mix asphalt or 0.20 foot of conventional asphalt binder or other approved modified asphalt binder mix. A 0.20-foot overlay of rubberized hot mix asphalt may be appropriate in certain circumstances and may be utilized with the concurrence of the Headquarters Program Advisor in the Headquarters Division of Maintenance – Pavement Program.

For flexible pavement CAPM projects with an IRI greater than 170 inches per mile, the standard design is to place a 0.25-foot hot mix asphalt overlay in two lifts Existing pavement may be milled or cold planed down to the depth of the overlay prior to placing the overlay. Situations where milling or cold planing may be beneficial or even necessary are to improve ride quality, maintain profile grade, maintain vertical clearance, or to taper (transition) to match an existing pavement or bridge surface.

If the necessary ride improvement cannot be adequately addressed with these CAPM treatments, the project should be developed as a roadway rehabilitation project.

A 0.06 foot – 0.10 foot non-structural wearing course (such as an open graded friction course) may be added, but is not to be considered part of the overlay requirements.

Deflection studies are not required for CAPM projects. The roadway rehabilitation requirements for overlays (see Index 635.1(1)) and preparation of existing pavement surface (Index 635.1(8)) apply to CAPM projects. Additional details and information regarding CAPM policies and strategies can be found in Design Information Bulletin 81 "Capital Preventive Maintenance Guidelines."

Topic 635 - Engineering Procedures for Flexible Pavement and Roadway Rehabilitation

635.1 Empirical Method

(1) *General.* The methods presented in this topic are based on studies for a ten-year pavement design life with extrapolations for twenty-year pavement design life. (For pavement design lives greater than twenty years, contact the Headquarters Office of Asphalt Pavement).

Because there are potential variations in materials and environment that could affect the performance of both the existing pavement and the rehabilitation strategy, it is difficult to develop precise and firm practices and procedures that cover all possibilities for the rehabilitation of pavements. Therefore, the pavement engineer should consult with the District Materials Engineer and other pertinent experts who are familiar with engineering, construction, materials, and maintenance of pavements in the geographical area of the project for additional requirements or limitations than those listed in this manual.

Rehabilitation strategies are divided into three categories:

- Overlay
- Mill and Overlay
- Remove and Replace

Rehabilitation designs are governed by one of the following three criteria:

- Structural adequacy
- Reflective crack retardation
- Ride quality

On overlay projects, the entire traveled way and paved shoulder shall be overlaid. Not only does this help provide a smoother finished surface, it also benefits bicyclists and pedestrians when they need to use the shoulder.

(2) *Data Collection.* Developing a rehabilitation strategy requires collecting background data as well as field data. The Pavement Condition Report (PCR), as-built plans, and traffic information are some of the sources used to prepare rehabilitation strategy recommendations. A thorough field investigation of the pavement surface condition, combined with a current deflection study and coring, knowledge of the subsurface conditions, thicknesses of existing flexible pavement layers, and a review of drainage conditions are all necessary for developing a set of appropriate rehabilitation strategies.

(3) *Deflection Studies.* Deflection studies along with coring data are used to measure the structural adequacy of the existing pavement. A deflection study is the process of selecting deflection test sections, measuring pavement surface deflection, and calculating statistical deflection values as described in California Test Method 356 for flexible pavement deflection measurements. A copy of the test method can be obtained and/or downloaded from the Department Pavement website.

To provide reliable rehabilitation strategies, deflection studies should be done no more than 18 months prior to the start of construction.

(a) Test Sections:

Test sections are portions of a roadway considered to be representative of roadway conditions being studied for rehabilitation. California Test Method 356 provides information on selecting test sections and different testing devices. Test sections should be determined in the field based on safe operation and true representation of pavement sections. Test sections can be determined either by the test operator or by the pavement engineer in the field.

Occasionally, a return to a project site may be required for additional testing after reviewing the initial deflection data in the office.

Individual deflection readings for each test section should be reviewed prior to determining statistical values. This review may locate possible areas that are not representative of the entire test section. An example would be a localized failure with a very high deflection. It may be more cost effective to repair the various failed sections prior to rehabilitation. Thus, the high deflection values in the repaired areas would not be included when calculating statistical values for the representative test sections.

(b) Mean and 80th Percentile Deflections:

The mean deflection level for a test section is determined by dividing the sum of individual deflection measurements by the number of the deflections:

$$\bar{x} = \frac{\Sigma D_i}{n}$$

Where:

$\bar{x}$ = mean deflection for a test section, in inches

D_i = an individual measured surface deflection in the test section, in inches

n = number of measurements in the test section

The 80th percentile deflection value represents a deflection level at which approximately 80 percent of all deflections are less than the calculated value and 20 percent are greater than the value. Therefore, a strategy based on 80th percentile deflection will provide thicker rehabilitation than using the mean value.

For simplicity, a normal distribution has been used to find the 80th percentile deflection using the following equation:

$$D_{80} = \bar{x} + 0.84s$$

Where:

D_{80} = 80th percentile of the measured surface deflections for a test section, in inches

s = standard deviation of all test points for a test section, in inches

$$s = \sqrt{\frac{\sum\left(D_i - \bar{x}\right)^2}{n-1}}$$

D_{80} is typically calculated as part of the deflection study done by the test operator. The pavement engineer should verify that the D_{80} results provided by the operator are accurate.

(c) Grouping:

Adjacent test sections may be grouped and analyzed together. There may be one or several groups within the project.

A group is a collection of test sections that have similar engineering parameters. Test sections can be grouped if they have all of the following conditions:

- Average D_{80} that vary less than 0.01 inch.
- Average existing hot mix asphalt thickness that vary less than 0.10 foot.
- Similar base material.
- Similar TI

Once groups have been identified, D_{80} and existing surface layer thickness of each group can be found by averaging the respective values of test sections within that group.

An alternative to the grouping method outlined above is to analyze each test section individually and then group them based on the results of analysis. This way, all the test sections that have similar rehabilitation strategies would fall into the same group.

(4) *Procedures for Rigid Pavement Overlay on Existing Flexible Pavement (Concrete Overlay).* For concrete overlay (sometimes referred to as whitetopping) strategies, only structural adequacy needs to be addressed. To address structural adequacy, use the tables in Index 623.1 to determine the thickness of the rigid layer. The overlay should be thick enough to be considered a structural layer. Therefore, thin or ultra thin concrete layers (< 0.65 foot) are not qualified as concrete overlay. To provide a smooth and level grade for the rigid surface layer, place a 0.10 foot to 0.15 foot HMA on top of the existing flexible layer.

(5) *Procedures for Flexible Overlay on Existing Flexible Pavement.*

(a) Structural Adequacy. Pavement condition, thickness of surface layer, measured deflections, and the projected TI provide the majority of the information used for determining structural adequacy. Structural adequacy is determined using the following procedures and rules:

(b) Determine the Tolerable Deflection at the Surface (TDS). The term “Tolerable Deflection” refers to the level beyond which repeated deflections of that magnitude produce fatigue failure prior to the planned TI. TDS is obtained from Table 635.1A by knowing the existing thickness of the flexible layer and TI. For existing flexible pavement over a treated

Table 635.1A
Tolerable Deflections at the Surface (TDS) in 0.001 inches

Exist. HMA thick (ft)	Traffic Index (TI)											
	5	6	7	8	9	10	11	12	13	14	15	16
0.00	66	51	41	34	29	25	22	19	17	15	14	13
0.05	61	47	38	31	27	23	20	18	16	14	13	12
0.10	57	44	35	29	25	21	19	16	15	13	12	11
0.15	53	41	33	27	23	20	17	15	14	12	11	10
0.20	49	38	31	25	21	18	16	14	13	12	10	10
0.25	46	35	28	24	20	17	15	13	12	11	10	9
0.30	43	33	27	22	19	16	14	12	11	10	9	8
0.35	40	31	25	20	17	15	13	12	10	9	8	8
0.40	37	29	23	19	16	14	12	11	10	9	8	7
0.45	35	27	21	18	15	13	11	10	9	8	7	7
0.50 (1)	32	25	20	17	14	12	11	9	8	8	7	6
TB (2)	27	21	17	14	12	10	9	8	7	6	6	5
	5.5	6.5	7.5	8.5	9.5	10.5	11.5	12.5	13.5	14.5	15.5	16.5
0.00	58	45	37	31	27	23	20	18	16	15	13	12
0.05	53	42	34	29	25	21	19	17	15	14	12	11
0.10	50	39	32	27	23	20	18	16	14	13	11	11
0.15	46	36	30	25	21	19	16	14	13	12	11	10
0.20	43	34	28	23	20	17	15	14	12	11	10	9
0.25	40	32	26	22	19	16	14	13	11	10	9	8
0.30	37	29	24	20	17	15	13	12	11	9	9	8
0.35	35	27	22	19	16	14	12	11	10	9	8	7
0.40	32	26	21	18	15	13	11	10	9	8	8	7
0.45	30	24	20	16	14	12	11	9	9	8	7	6
0.50 (1)	28	22	18	15	13	11	10	9	8	7	7	6
TB (2)	24	19	15	13	11	10	8	7	7	6	5	5

Notes:

(1) For an HMA thickness greater than 0.50 ft use the 0.50 ft depth.

(2) Use the TB (treated base) line to represent treated base materials, regardless of the thickness of HMA cover.

Table 635.1B

Gravel Equivalence Needed to Reduce Deflection

Percent Reduction In Deflection (PRD or PRM) (1)	GE (in feet) For HMA Overlay Design	Percent Reduction In Deflection (PRD or PRM) (1)	GE (in feet) For HMA Overlay Design
5	0.02	46	0.55
6	0.02	47	0.57
7	0.02	48	0.59
8	0.02	49	0.61
9	0.03	50	0.63
10	0.03	51	0.66
11	0.04	52	0.68
12	0.05	53	0.70
13	0.05	54	0.72
14	0.06	55	0.74
15	0.07	56	0.76
16	0.08	57	0.79
17	0.09	58	0.81
18	0.09	59	0.83
19	0.10	60	0.85
20	0.11	61	0.87
21	0.12	62	0.89
22	0.14	63	0.91
23	0.15	64	0.94
24	0.16	65	0.96
25	0.18	66	0.98
26	0.19	67	1.00
27	0.20	68	1.02
28	0.21	69	1.04
29	0.23	70	1.06
30	0.24	71	1.09
31	0.26	72	1.11
32	0.28	73	1.13
33	0.29	74	1.15
34	0.31	75	1.17
35	0.33	76	1.19
36	0.35	77	1.22
37	0.37	78	1.24
38	0.38	79	1.26
39	0.40	80	1.28
40	0.42	81	1.30
41	0.44	82	1.32
42	0.46	83	1.34
43	0.48	84	1.37
44	0.51	85	1.39
45	0.53	86	1.41

Note: (1) PRD is Percent Reduction in Deflection at the surface.
PRM is Percent Reduction in deflection at the Milled depth.

base, use TI and the TDS values in the row for Treated Base (TB) found in Table 635.1A

The existing base is considered treated if it meets all of the following conditions:

- Its depth is equal to or greater than 0.35 foot.
- The D_{80} is less than 0.015 inch.

(1) It is rigid pavement, Lean Concrete Base (LCB), or Class A Cement Treated Base (CTB-A). For each group compare the TDS to the average D_{80}. The D_{80} is the 80th percentile deflection value. It represents a deflection level at which approximately 80 percent of all deflections of a sample group are less than the calculated value and 20 percent are greater than the value. Therefore, a strategy based on the 80th percentile deflection will provide thicker rehabilitation than using the mean deflection.

If the average D_{80} is greater than the TDS, determine the required percent reduction in deflection at the surface (PRD) to restore structural adequacy as follows:

$$\text{PRD} = \frac{AverageD_{80} - TDS}{AverageD_{80}}(100)$$

Where:

PRD = Percent Reduction in Deflection required at the surface, as percent

TDS = Tolerable Deflection at the Surface, in inches

Average D_{80} = mean of the 80th percentile of the deflections for each group, in inches

(2) Using the calculated PRD and Table 635.1B, determine the GE required to reduce the deflections to less than the tolerable level.

(3) Divide the GE obtained from Table 635.1B by the appropriate G_f for the overlay material to determine the required thickness of the overlay.

$$\text{Thickness (t)} = \frac{GE}{G_f}$$

Commonly used G_f for flexible pavement rehabilitation are presented in Table 635.1C.

Table 635.1C

Commonly Used G_f for Flexible Pavement Rehabilitation

Material	G_f [(1)]
Hot Mix Asphalt Overlay	1.9
Hot Recycled Asphalt	1.9
Cold in-Place Recycled Asphalt	1.5
HMA Below the Analytical Depth [(2)]	1.4

NOTES:

(1) For G_f of bases and subbases see Table 663.1B.

(2) Analytical depth is defined in 635.1(6)(a).

(c) Reflective Cracking. The goal of these procedures is to keep cracks at the bottom of the surface course layer from propagating to the surface during the pavement design life. Retarding the propagation of cracks is an important factor to consider when engineering flexible pavement overlays. The procedures and rules for engineering for reflective cracking retardation are as follows:

(1) Determine the minimum thickness required for a 10-year pavement

design life. For flexible pavements over untreated bases, the minimum thickness of a HMA overlay with a ten-year design life should be half the thickness of the existing surface course layer but not to exceed 0.35 foot.

For flexible pavements over treated bases (as defined in the previous section on structural adequacy), a minimum HMA overlay of 0.35 foot should be used for a 20-year design life.

Exception: when the underlying material is a thick rigid layer (0.65 foot or more) such as an overlaid jointed plain concrete pavement that was not cracked and seated, a minimum thickness of 0.45 foot should be used.

(2) Adjust thickness if the pavement design life is different than 10 years. For a twenty-year design life, experience has shown that the thickness should be 125 percent of the ten-year thickness for reflective cracking retardation.

(3) Adjust overlay thickness for alternative materials.

A thickness equivalency of not more than 1:2 is given to the RHMA-G when compared to the HMA for reflective crack retardation. The equivalencies are tabulated in Table 635.1D.

If a SAMI-R is placed under a non-rubberized hot mix asphalt that is engineered for reflective crack retardation, the equivalence of a SAMI-R depends upon the type of base material under the existing pavement. When the base is a treated material, a SAMI-R placed under HMA or OGFC is considered to be equivalent to 0.10 foot of HMA. When the base is an untreated material SAMI-R is equivalent to 0.15 foot of HMA.

Table 635.1D Reflective Crack Retardation Equivalencies (Thickness in feet)

HMA (1)	RHMA-G	RHMA-G over SAMI-R
0.15	0.10	
0.20	0.10	
0.25	0.15	
0.30	0.15	
0.35	• 0.15 if crack width < 1/8 inch • 0.20 if crack width ≥ 1/8 inch or underlying material CTB, LCB, or rigid pavement	• N/A for crack width < 1/8 inch • 0.10 if crack width ≥ 1/8 inch and underlying material untreated • 0.15 if crack width ≥ 1/8 inch and underlying material CTB, LCB, or rigid pavement
0.45	0.15 over 0.15 HMA	0.20

NOTE:

(1) See Index 635.1(5)(b) for minimum and maximum HMA thicknesses recommended by the Department for reflective crack retardation on flexible pavements.

A Geosynthetic Pavement Interlay (GPI) placed under HMA that is engineered for reflective crack retardation provides the equivalent of 0.10 foot of HMA. This allows the engineer to decrease the new profile grade and also save on HMA materials.

Wearing courses are not included in the thickness used to address reflective cracking.

Thicker sections may be warranted. Factors to be considered that might necessitate a thicker overlay are:

- Type, sizes, and amounts of surface cracks.
- Extent of localized failures.
- Existing performance material and age.
- Thickness and performance of previous rehabilitation strategy.
- Environmental factors.
- Anticipated future traffic loads (Traffic Index).

As always, sound engineering judgment will be necessary for final decisions. Final decision for when to use more than the minimum requirements found in this manual rests with the District.

(d) Ride Quality. Ride quality is evaluated based on the pavement's smoothness. The Department records smoothness as part of Pavement Condition Survey using the International Roughness Index (IRI). According to FHWA, the IRI value that most motorists consider uncomfortable for flexible pavement is 170 inches per mile. When IRI measurements are 170 inches per mile or greater, the engineer must address ride quality.

(e) To improve ride quality, place a hot mix asphalt overlay thick enough (0.25 foot minimum) to be placed in two lifts. RHMA-G may be placed in two 0.10 foot lifts to meet the ride quality requirement. However, if a 0.10 foot layer cools prior to compaction, this strategy is inappropriate. A wearing course may be included in the ride quality thickness. SAMI's do not have any effect on ride quality.

Ride quality will ultimately govern the rehabilitation strategy if the requirements for structural adequacy and reflective crack retardation are less than 0.25 foot.

Note that the Standard Specifications require the Contractor to place a 0.25 foot HMA in one layer. Projects with rehabilitation recommendations based on improving ride quality must specify in the Special Provisions that the overlay needs to be placed in two lifts. Examples of design calculations for flexible overlay thickness on existing flexible pavement are available on the Department Pavement website.

(6) *Mill and Overlay Procedures.* Mill and Overlay is the removal of part of the surface course and placement of an overlay. Since existing pavement thicknesses will have slight variations throughout the project length, leave at least the bottom 0.15 foot of the existing surface course intact to ensure the milling machine does not loosen the base material or contaminate the recycled mix during the hot or cold in-place recycling. If removal of the entire surface course layer and any portion of the base are required, use the procedures for Remove and Replace in Index 635.1(7).

a) Structural Adequacy. The engineering procedures for determining the structural adequacy for Mill and Overlay are the same as those for overlays found in Index 635.1(1), with the exception of the following:

- TDS is determined using the thickness of the existing pavement prior to milling.
- Deflections are measured at the surface and adjusted to the milled depth.

The Engineer must consider milling down to the "analytical depth". As defined by the Department, the analytical depth is the least of:

- The milled depth where the Percent Reduction in deflection required at the Milled depth (PRM) reaches 70 percent.
- The milled depth equals 0.50 foot.

- The bottom of the existing HMA layer.

The percent reduction in deflection required at the milled depth is based research that determined deflections increase by 12 percent for each additional 0.10 foot of milled depth up to the analytical depth. Once the analytical depth is reached, the existing HMA material below is considered to be of questionable structural integrity and hence is assigned a G_f of 1.4. Since it is not known at what milled depth the 70 percent PRM level or analytical depth will be reached, an iterative type of calculation is required.

Using the thickness of the existing HMA layer, the TI, and base material, determine the TDS from Table 635.1A. The deflection at the milled depth is found from the equation:

$$\mathrm{DM} = \mathrm{D}_{80} + \left[(12\%) \left(\frac{\mathit{MillDepth}}{\mathit{0.10\ ft}} \right) (\mathrm{D}_{80}) \right]$$

Where

D_{80} = 80th Percentile deflections, in inches.

Mill Depth = the depth of the milling in feet.

DM = the calculated deflection at the Milled depth in inches.

Then:

$$PRM = \left(\frac{DM - TDS}{DM} \right) (100)$$

Where

PRM = Percent Reduction in deflection required at the Milled depth.

TDS = Tolerable Deflection at the Surface in inches.

Utilizing the calculated PRM value, go to Table 635.1B to get the total GE required to be placed on top of the milled pavement surface. The total GE required to reduce the measured deflection to the tolerable level is a combination of:

- The GE determined from the overlay calculations.
- The GE required to replace the material removed by the milling process.

If the milling goes below the analytical depth, the Additional GE that is required to replace the existing HMA below the analytical depth is calculated by multiplying the G_f of 1.4 by the milled depth below the analytical depth:

Additional GE = [(1.4)(milled depth below the analytical depth)]

To determine the total GE for the overlay, the Additional GE below the analytical depth is added to the required GE above the analytical depth (found from Table 635.1B). As stated in Index 633.1(1)(d), the required minimum thickness of the overlay is determined by dividing the total GE by the G_f of the new overlay material.

$$\text{Thickness (t)} = \frac{\mathrm{GE}}{\mathrm{G_f}}$$

If milled material is to be replaced by Hot Recycled Asphalt (HRA), the overlay thickness is the same as that of HMA since both materials have a G_f of 1.9 (see Table 635.1C).

Since Cold In-Place Recycled Asphalt (CIR) has low resistance to abrasion, if the milled material is to be replaced with CIR, the CIR layer must be covered with a wearing surface shortly after the recycling process. To determine the required thickness of the cap layer, first determine the GE of the CIR layer:

$$\mathrm{GE_{CIR}} = (\text{CIR thickness})(\mathrm{G_{f\ CIR}})$$

Where:

GE_{CIR} = Gravel Equivalence of the CIR

$G_{f\ CIR}$ = Gravel Factor of CIR = 1.5 (see Table 635.1C)

The thickness of the cap layer is determined as follows:

$$\text{Cap Layer Thickness} = \frac{GE_{TOTAL} - GE_{CIR}}{G_f}$$

Where:

GE_{TOTAL} = Total GE requirement of CIR and cap layers.

G_f = Gravel Factor of the cap material.

If the cap layer is OGFC, its thickness should not be considered in pavement structure design. It is recommended to round up to get the CIR and cap layer thicknesses.

(b) Reflective Cracking. The minimum thickness for reflective cracking is determined using the same procedures used for reflective cracking for overlays found in Index 635.1(5)(b) except that the thickness is determined based on the remaining surface layer rather than the initial surface layer.

(c) Ride Quality. Milling the existing surface and overlaying with new surface course is considered sufficient to smooth a rough pavement.

(7) *Remove and Replace.* The Remove and Replace operation consists of removing the entire surface layer and part or all of the base and subbase material. The entire removed depth is then replaced with a new flexible or rigid pavement structure. The Remove and Replace strategy is most often used when:

- It is not possible to maintain the existing profile grade using Mill and Overlay.
- Existing base and or subbase material is failing and needs to be replaced.
- It is the most cost effective strategy based on life cycle cost analysis.

Remove and Replace covers a variety of strategies. The discussion found here provides some general rules and minimum requirements for Remove and Replace strategies in general. For more specific information see the technical guidance on the Department Pavement website.

Because the existing surface layer is removed only structural adequacy needs to be addressed for Remove and Replace.

(a) Partial Depth Removal. When only a portion of the existing depth is being removed, consideration needs to be given to the strength of the remaining pavement structure. Because the pavement has been stressed and has been subject to contamination from fines and other materials over time, it does not have the same strength (GE) as new material. Currently, for partial depth removals, the most effective engineering method is to determine the theoretical deflection of the remaining material otherwise known as DM. It should be noted that the greater the depth of removal, the less accurate the determination might be of the calculated deflections.

Also, using deflections for Remove and Replace strategies is also less accurate if a bulldozer or a scraper is used to remove the material under the pavement instead of a milling machine. This method of removing material disturbs the integrity of the in-place material from which the deflections were measured.

Because of these issues, the DME may require reduced GE from what is found in this manual or additional pavement thickness. Final determination of what GE is used rests with the District.

It is recommended that if the removal depth is more than 1 foot, determine the pavement thickness and layers use the method for new or reconstructed pavements discussed in Index 633.1. If the pavement structure is being replaced with rigid pavement, the resulting total pavement structure (including existing pavement left in place) cannot be less than the minimum values found in the rigid pavement catalog in Topic 623.

The analysis used for partial depth Remove and Replace with flexible pavement is similar to the Mill and

Overlay analysis. The procedures are as follows:

(1) Consider milling down to what is called the analytical depth. This is an iterative type of calculation since it is not known at what milling depth the analytical depth will be reached.

(2) Use the thickness of the existing HMA layer, the design TI and base material in Table 635.1A to determine the TDS. Then find the DM knowing D_{80} and the mill depth. Use DM and TDS to find the percent reduction in deflection at the milled depth (PRM).

(3) Utilizing this calculated PRM value go to Table 635.1B to obtain the GE required to be placed on top of the milled surface. When the milled depth reaches the analytical depth, the analysis changes. The GE for the material milled below the analytical depth is added to the GE required at the analytical depth. The GE for each layer is calculated by multiplying G_f by the thickness of the layer milled.

(4) Determine the required minimum thickness of HMA needed by dividing the sum of the GE's by the G_f of the new HMA (see equation below.)

$$\text{Thickness (t)} = \frac{\text{GE}}{\text{G}_f}$$

For the Remove and Replace method, use the G_f for the new HMA commensurate with the TI and HMA thickness found in Table 633.1. The total HMA thickness can be solved for each 0.05 foot of material milled until the desired profile is reached. Round the replacement thickness to the nearest 0.05 foot.

(5) Adjust thicknesses as needed for alternate materials.

(b) Full depth removal. When material is removed all the way to the subgrade, the Remove and Replace strategy should be engineered using the same procedures used for new construction found in Index 633.1.

(8) *Preparation of Existing Pavement.* Existing pavement distresses should be repaired before overlaying the pavement. Cracks wider than ¼ inch should be sealed; loose pavement removed/replaced; and potholes and localized failures repaired. Undesirable material such as bleeding seal coats or excessive crack sealant should be removed before paving. Existing thermoplastic traffic striping and raised pavement markers should also be removed. Routing cracks before applying crack sealant has been found to be beneficial. The width of the routing should be ¼ inch wider than the crack width. The depth should be equal to the width of the routing plus ¼ inch. In order to alleviate the potential bump in the overlay from the crack sealant, leave the crack sealant ¼ inch below grade to allow for expansion (i.e., recess fill). The Materials Report should include a reminder of these preparations. Additional discussion of repairing existing pavement can be found on the Department Pavement website.

(9) *Choosing the Rehabilitation Strategy.* The final strategy should be chosen based on pavement life-cycle cost analysis (LCCA). The strategy should also meet other considerations such as constructibility, maintenance, and the other requirements found in Chapter 610.

635.2 Mechanistic-Empirical Method

For information on Mechanistic-Empirical Design application and requirements, see Index 606.3.

Topic 636 - Other Considerations

636.1 Traveled Way

(1) *Mainline.* No additional considerations.

(2) *Ramps and Connectors.* Rigid pavement should be considered for freeway-to-freeway connectors and ramps near major commercial or industrial areas (TI > 14.0), truck terminals, and all truck weighing and inspection facilities.

(3) *Ramp Termini*. Distress is compounded on flexible pavement ramp termini by the dissolving action of oil drippings combined with the braking of trucks. Separate pavement strategies should be developed for these ramps that may include thicker pavement structures, special asphalt binders, aggregate sizes, or mix designs. Rigid pavement should be considered for exit ramp termini where there is a potential for shoving or rutting. At a minimum, rigid pavement should be used for exit ramp termini of flexible pavement ramps where a significant volume of trucks is anticipated (TI > 12.0). For the engineering of rigid pavement ramp termini, see Index 626.1(3).

636.2 Shoulders

The TI for shoulders is given in Index 613.5(2). See Index 1003.5(1) for surface quality guidance for bicyclists.

636.3 Intersections

Where intersections have "STOP" control or traffic signals, special attention is needed to the engineering of flexible pavements to minimize shoving and rutting of the surface caused by trucks braking, and early failure of detector loops. Separate pavement strategies should be developed for these intersections that may include thicker pavement structures, special asphalt binders, aggregate sizes, or mix designs. Rigid pavement is another alternative for these locations. For additional information see Index 626.3. For further assistance on this subject, the Design Engineer should contact the District Materials Engineer, or Headquarters Division of Maintenance – Pavement Program.

636.4 Roadside Facilities

(1) *Safety Roadside Rest Areas.* Safety factors for the empirical method should be applied to the ramp pavement but not for the other areas.

For truck parking areas, where pavement will be subjected to truck starting/stopping and oil drippings which can soften asphalt binders, separate flexible pavement structures which may include thicker structural sections, alternative asphalt binders, aggregate sizes, or mix designs should be considered. Rigid pavement should also be considered.

(2) *Park & Ride Facilities.* To engineer a park and ride facility based on the standard traffic projections is not practicable because of the unpredictability of traffic. Therefore, standard structures, based on anticipated typical load, have been adopted. However, if project site-specific traffic information is available, it should be used with the standard engineering procedures.

The layer thicknesses shown in Table 636.4 are based on previous practices. These pavement structures are minimal, but are considered adequate since additional flexible surfacing can be added later, if needed, without the exposure to traffic or traffic-handling problems typically encountered on a roadway.

(3) *Bus pads.* Use rigid or composite pavement strategies for bus pads.

Table 636.4
Pavement Structures for Park and Ride Facilities

California R-value for the Subgrade Soil	Thickness of Layers	
	HMA[1] (ft)	AB (ft)
< 40	0.25	0
But < 60	0.15	0.35
≥ 40	0.15	0
≥ 60	Penetration Treatment[2]	

NOTES:

(1) Place in one lift.

(2) Penetration Treatment is the application of a liquid asphalt or dust palliative on compacted roadbed material. See Standard Specifications.

Topic 637 - Engineering Analysis Software

Software programs for engineering flexible pavements using the procedures in this chapter can be found on the Department Pavement website. These programs employ the procedures and requirements for flexible pavement engineering enabling the engineer to compare numerous combinations of materials in seeking the most cost effective pavement structure.

CHAPTER 640
COMPOSITE PAVEMENTS

Topic 641 - Types of Composite Pavement

Index 641.1 - Asphalt Over Concrete Composite Pavement

This configuration consists of an asphalt layer over concrete surface layer (typically jointed plain concrete pavement or continuous reinforced concrete pavement) where the asphalt layer is used to protect or enhance the performance of the concrete pavement. (Asphalt layers over lean concrete base or cement treated base are considered to be flexible pavements for the purposes of this manual.) The function of the asphalt layer is to act as a thermal and moisture blanket to reduce the vertical temperature and moisture gradient within the concrete surface layer and decrease the deformation (curling and warping) of concrete slabs. In addition, the asphalt layer acts as a wearing course to reduce wearing effect of wheel loads on the concrete surface layer.

Asphalt over concrete composite pavements are found most often on older pavements that have had asphalt overlay such as hot mix asphalt, open graded friction course, or rubberized hot mix asphalt, placed over previously built jointed plain concrete pavement (JPCP) or continuously reinforced concrete pavement (CRCP.) New or reconstructed composite pavements with asphalt layer over JPCP or CRCP typically have not been built in the past on State highways because they have been viewed as combining the disadvantages of rigid pavements (higher initial cost) and flexible pavements (more frequent maintenance).

Thin flexible layers (i.e. sacrificial wearing course) have sometimes been placed over JPCP or CRCP to improve ride quality or friction of the rigid layer. Because ride quality and friction can also be improved by grooving or diamond grinding the existing concrete layer, the Engineer should perform a life-cycle cost analysis (LCCA) to determine if diamond grinding/grooving or an asphalt nonstructural overlay is more cost effective before deciding which option to select.

Some cases in which the asphalt over concrete composite pavement option is used include:

- To match the existing pavement structure when widening;
- When adding truck lanes to an adjacent flexible pavement;
- To provide a nonstructural surface course to an existing rigid pavement that is still structurally sound but is worn out on the surface.

641.2 Concrete Over Asphalt Composite Pavement

Because of the minimum 0.70 foot thickness requirements for concrete surface course, all pavements with concrete surface course are engineered according to the standards and procedures for rigid pavements in Chapter 620.

Topic 642 - Engineering Criteria

642.1 Engineering Properties

The engineering properties found in Index 622.1 for rigid pavement and Index 632.1 for flexible pavement apply to composite pavements. Care should be taken in selecting materials in the asphalt layer to resist reflective crack propagation from the underlying concrete layer and facilitate construction of generally thin asphalt layers.

642.2 Performance Factors

Flexible layers placed over rigid surface layers need to be engineered and use materials that will meet the following requirements:

(1) *Reflective Cracking*. Joints or cracks from the underlying concrete surface layer should not reflect through the asphalt layer for the service life of the composite pavement.

(2) *Smoothness*. The asphalt layer should be engineered to provide an initial IRI of 60 inches per mile and maintain an IRI that is less than 170 inches per mile throughout its service life.

(3) *Bonding.* A major factor in the effectiveness and service life of the composite pavement is the condition of the bond between the asphalt and concrete layers. For a good bond, the

thickness of the asphalt layer does not play an important role in its service life.

Therefore, for practical purposes, if there is no thickness requirement from the structural/constructibility point of view, the minimum thickness of the asphalt layer should be based on material factors such as, gradation and aggregate structure, type of binder, etc. To achieve the maximum bond between asphalt and concrete layers, consult the District Materials Engineer or Headquarters Office of Asphalt Pavement for options on effective bonding methods.

642.3 Overlay Limits

On overlay projects, the entire traveled way and paved shoulder shall be overlaid. Not only does this help provide a smoother finished surface, it also benefits bicyclists and pedestrians when they need to use the shoulder.

Topic 643 - Engineering Procedures for New Construction and Reconstruction

643.1 Empirical Method

Before deciding to construct a new composite pavement, a LCCA should be completed to determine whether the composite pavement is more cost effective over the long term than asphalt or concrete pavement alternatives.

At present, there is no comprehensive procedure to engineer a structural layer of asphalt surface course over a concrete surface course layer of JPCP or CRCP. Research is under way to provide guidelines for engineering and construction of composite pavements. When engineering composite pavements using JPCP or CRCP, the rigid pavement structure is engineered using the procedures in Index 623.1. No reduction is made to the thickness of the concrete layer on account of the asphalt overlay layer. The asphalt pavement is treated as a nonstructural wearing course, and thus has no structural value.

When enough information is not available, the thickness requirement for placing an asphalt layer over an existing rigid pavement can be used as a conservative thickness for a new pavement. See Index 625.1 for further details.

643.2 Mechanistic-Empirical Method

For engineering an asphalt on concrete composite pavement using Mechanistic-Empirical Design follow the procedures and requirements in Index 606.3 and 633.2.

Topic 644 - Engineering Procedures for Pavement Preservation

644.1 Preventive Maintenance

Preventive Maintenance is used to maintain the asphalt surface course layer or to replace thin asphalt layers (i.e., non-structural wearing courses) placed over concrete surface course layer. If work is needed to repair the underlying concrete layer, it should be developed as a CAPM (Index 644.2) or roadway rehabilitation (Topic 645) project. Additional information on preventive maintenance of the asphalt layer of a composite pavement is the same as for the flexible pavements, which can be found in the "Maintenance Technical Advisory Guide (MTAG)" available on the Department Pavement website.

644.2 Capital Preventive Maintenance (CAPM)

The CAPM warrants for concrete and asphalt pavements in Index 624.2 and 634.2 apply to composite pavements. The procedures and designs for asphalt over concrete composite pavement CAPM projects are the same as those for flexible pavements (see Index 634.2) except digouts may require concrete slab replacement and/or base repair. In the case of previously constructed crack, seat, and asphalt overlay projects, it may be beneficial to mill a portion of the existing asphalt layer prior to overlaying. Milling will reduce the thickness of the existing cracked pavement and therefore provide added life to the overlay.

The roadway rehabilitation requirements for overlays (see Index 645.1) and preparation of existing pavement surface (Index 645.1(3)) also apply to CAPM projects. Additional details and information regarding CAPM policies and strategies can be found in Index 603.3, PDPM

Appendix H, and Design Information Bulletin 81 "Capital Preventive Maintenance Guidelines."

Topic 645 - Engineering Procedures for Pavement Rehabilitation

645.1 Empirical Method

Procedures for engineering rehabilitation projects for asphalt over concrete composite pavement using empirical methods are as follows:

Because the asphalt surface layer is considered to have no structural value, only reflective cracking and ride quality need to be considered.

(1) *Reflective cracking*. If the asphalt layer is placed over an existing concrete pavement, the thickness is calculated based on the procedure outlined for rigid pavement rehabilitation. The thickness depends on the design life of asphalt surface course, as well as mix gradation, type and percentage of the binder.

   For additional information on rehabilitation of composite pavement with rigid surface courses refer to the Concrete Pavement Guide available on the Department Pavement website.

(2) *Ride Quality*. When the smoothness of the existing roadway is 170 inches per mile or greater as measured by the International Ride Index (IRI), a minimum 0.25 foot consisting of 0.10 foot HMA (leveling course) followed by a minimum of 0.15 foot HMA or RHMA surface course layer. A nonstructural wearing course may be placed on top lift. Pavement interlayers between the leveling course and surface course may also be considered. Note that in some cases, existing pavement will need to be repaired to assure the roadway smoothness will remain below 170 inches per mile throughout the life of the overlay.

(3) *Preparation for Placing Asphalt Layer Over Existing Concrete Pavement.* Existing pavement distresses should be repaired before overlaying the pavement. Cracks wider than 3/8 inch should be sealed or repaired. Undesirable material such as bleeding seal coats or excessive crack sealant should be removed before paving. Existing thermoplastic traffic striping and raised pavement markers also should be removed. Spalls in rigid pavement should be repaired and broken slabs or punchouts replaced. Grind existing concrete pavement as needed to eliminate rough ride and faulting. Consider dowel bar retrofit when it will help keep faulting from re-emerging. Loose asphalt wearing course should be removed and replaced, and potholes and localized failures repaired. Ideally, existing non-structural wearing courses should be removed and, if needed, underlying pavement repaired prior to placing a new asphalt wearing course. In some cases it may be more practical to overlay over the existing layer. (A LCCA of the two options will help determine which of these options is more cost effective. Note that when doing a LCCA, the need to ultimately remove asphalt layers in the future should be identified and included in the costs for the analysis.)

645.2 Mechanistic-Empirical Method

For information on Mechanistic-Empirical Design and requirements, see Index 606.3.

CHAPTER 650
PAVEMENT DRAINAGE

Topic 651 - General Considerations

Index 651.1 - Impacts of Drainage on Pavement

Saturation of the pavement or underlying subgrade, or both, generally results in a decrease in strength or ability to support heavy axle loads. Potential problems associated with saturation of the structural section and subgrade include:

- Pumping action.
- Differential expansion (swelling) of expansive subgrade.
- Frost damage in freeze-thaw areas.
- Erosion and piping of fine materials creating voids which result in the loss of subgrade support.
- Icing of pavement surface from upward seepage.
- Stripping of asphalt concrete aggregates.
- Accelerated oxidation of asphalt binder.

Water can enter the pavement as surface water through cracks, joints, and pavement infiltration, and as groundwater from an intercepted aquifer, a high water table, or a localized spring. These sources of water should be considered and provisions should be made to handle both. The pavement structure drainage system, which is engineered to handle surface water inflow, is generally separated from the subsurface drainage system that is engineered to accommodate encroaching subsurface water. This chapter covers surface water drainage while the subsurface drainage system is covered in Chapter 840.

651.2 Drainage System Components and Requirements

The basic components of a pavement structural section drainage system are:

(1) *Drainage Layer.* A treated permeable base (TPB) drainage layer may be useful where it is necessary to drain water beneath the pavement. A TPB requires the use of edge drains or some other method of draining water out and away from the pavement; otherwise the collected water will become trapped. If a TPB drainage layer is used, it should be placed immediately below the surface layer for interception of surface water that enters the pavement. The drainage layer limits are shown in Figure 651.2A. Further information for TPB can be found in Index 662.3.

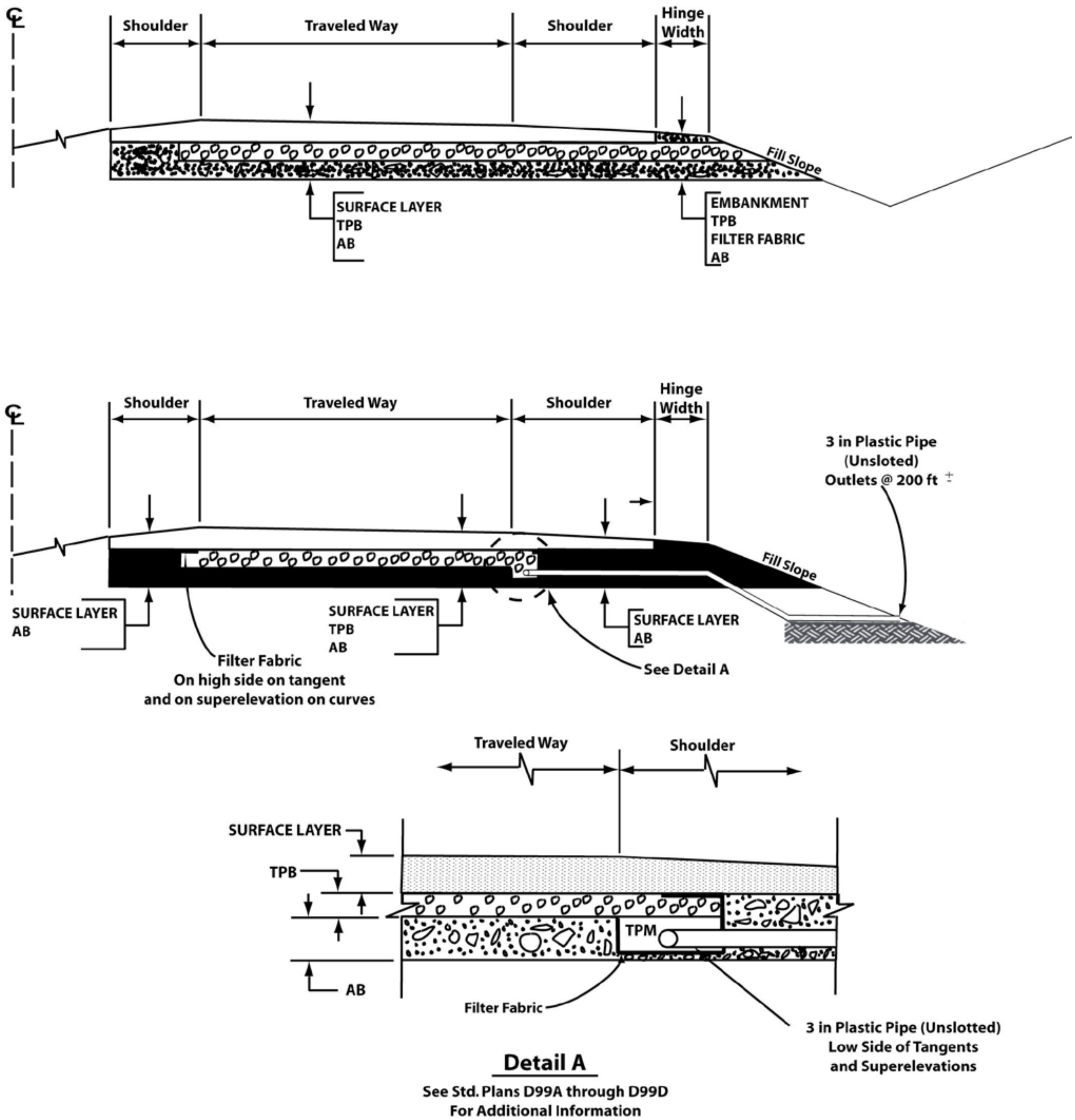
When there is concern that the infiltrating surface water may saturate and soften the underlying subbase or subgrade (due either to exposure during construction operations or under service conditions), a filter fabric or other suitable membrane should be utilized and applied to the base, subbase, or subgrade on which the TPB layer is placed to prevent migration of fines and contamination of the TPB layer by the underlying material.

When using TPB, special attention should be given to drainage details wherever water flowing in the TPB encounters impermeable abutting pavement layers, a structure approach slab, a sleeper slab, a pavement end anchor/transition, or a pressure relief joint. In any of these cases, a cross drain interceptor should be provided. Details of cross drain interceptors at various locations are shown in Figure 651.2B. The cross drain outlets should be tied into the longitudinal edge drain collector and outlet system with provision for maintenance access to allow cleaning.

In some situations, underground water from landscape irrigation or other sources may tend to saturate the existing slow-draining layers, thereby creating the potential for pumping and pavement damage. In this case, the pavement should be engineered to provide for removal of such water when reconstruction is required.

(2) *Collector System.* If constraints exist or where it is not practical to drain water out of the pavement by other means, a collector system should be provided to drain water from the drainage layer. Collector systems include a 3-inch slotted plastic pipe edge drain installed in a longitudinal collector trench as shown in Figure 651.2A. In areas where the profile grade

Figure 651.2A
Typical Section with Treated Permeable Base Drainage Layer



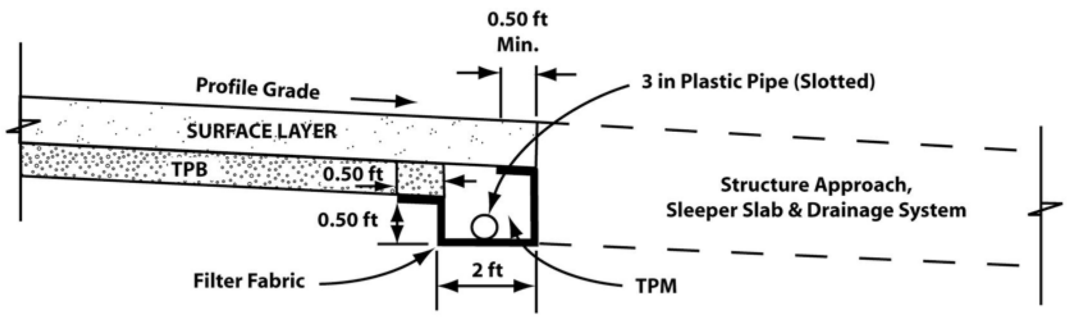
NOTES:

(1) Section shown is a half-section of a divided highway. An edge drain collector and outlet system should be provided if insufficient Right of Way precludes a retention basin.

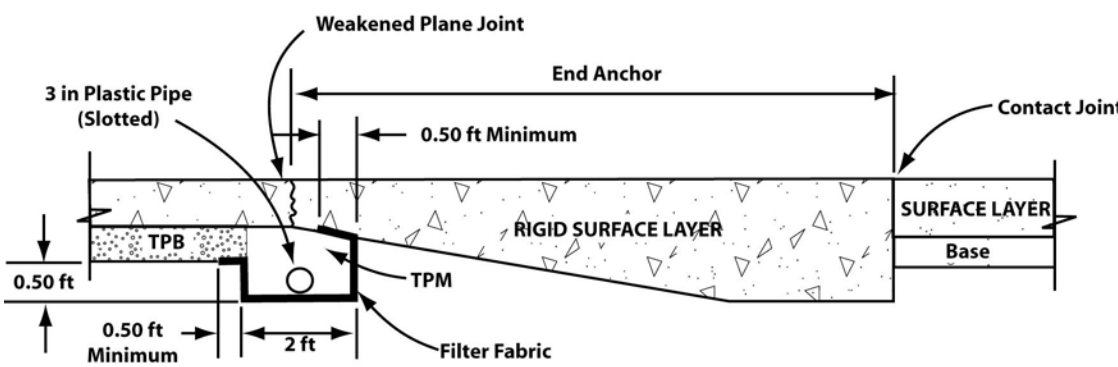
(2) This figure is only intended to show typical pavement details, for geometric cross section details, see Chapter 300.

Figure 651.2B

Cross Drain Interceptor Details For Use with Treated Permeable Base



AT STRUCTURE APPROACH
(LONGITUDINAL SECTION)



AT END ANCHOR
(LONGITUDINAL SECTION)

is equal to or greater than 4 percent, intermediate cross drain interceptors, as shown in Figure 651.2C should be provided at an approximate spacing of 500 feet. This will limit the longitudinal seepage distance in the drainage layer, minimizing the drainage time and preventing the buildup of a hydrostatic head under the surface layer. Cross drain interceptor trenches must be sloped to drain.

In addition, cross drains need to be provided at the low-end terminal of TPB projects, as shown in Figure 651.2C. Care should be taken to coordinate the cross drains with the longitudinal structural section drainage system. Drainage layers in roadway intersections and interchanges may require additional collector trenches, pipes, and outlets to assure rapid drainage of the pavement.

A standard longitudinal collector trench width of 1 foot has been adopted for new construction to accommodate compaction and consolidation of the TPB alongside and above the 3-inch slotted plastic pipe.

When a superelevation cross slope begins to drain the water through the TPB to the low side of pavement in cut sections, an edge drain system may be considered to direct water to an area where ponding will not occur.

(3) *Outlets, Vents, and Cleanouts.* Pavements should be engineered to promote free drainage whenever applicable. Alternative strategies are provided, as shown in Figure 651.2A. Incorporation of a TPB daylighting to the edge of embankment may be considered; otherwise, an edge drain collector and outlet system may provide positive drainage of the structural section.

When edge drains are used, plastic pipe (unslotted) outlets should be provided at proper intervals for the pavement drainage system to be free draining. The spacing of outlets (including vents and cleanouts) should be approximately 200 feet (250 feet maximum). Outlets should be placed on the low side of superelevations or blockages such as bridge structures.

The trench for the outlet pipe must be backfilled with material of low permeability, or provided with a cut-off wall or diaphragm, to prevent piping.

The outlets must be daylighted, connected to culverts or drainage structures, or discharged into gutters or drainage ditches. The area under the exposed end of a daylighted outlet should have a splash block or be paved to prevent erosion and the growth of vegetation, which will impede flows from the outlet. Ready access to outlets, and the provision of intervening cleanouts when outlet spacing exceeds a maximum distance of 250 feet, should be provided to facilitate cleaning of the pavement drainage system. Typical details are shown on the Standard Plans for Edge Drain Outlet and Vent Details.

The end of each outlet pipe should be indicated by an appropriate marker to facilitate location and identification for maintenance purposes and to reduce the likelihood of damage by vehicles and equipment. Consult the District Division of Maintenance for the preferred method of identification.

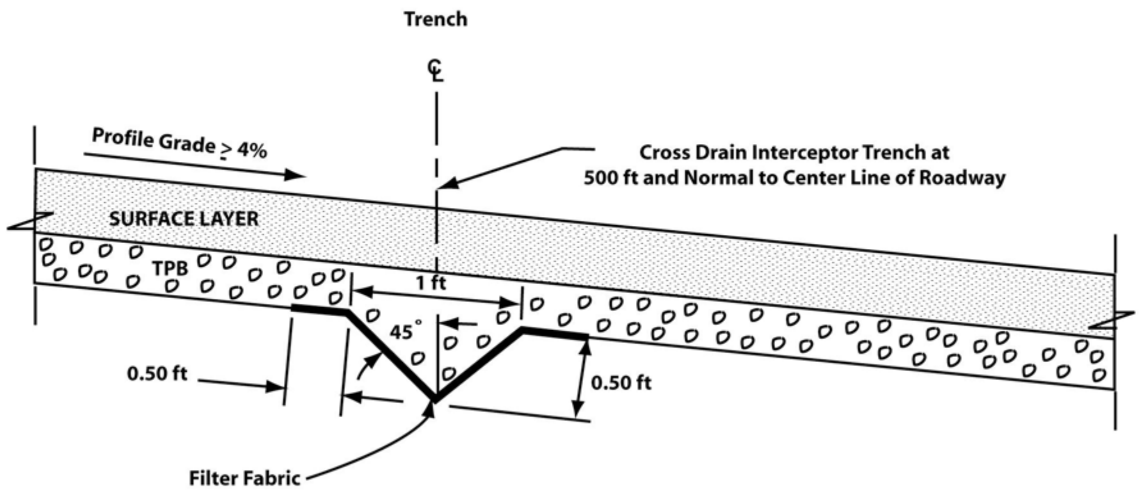
(4) *Filter Fabric.* Filter fabric should be placed as shown in Figures 651.2A and B, respectively, to provide protection against clogging of the treated permeable material (TPM) by intrusion of fines. Filter fabric should be selected based upon project specific materials conditions to ensure continuous flow of water and preclude clogging of the filter fabric openings. Consult with the District Materials Engineer to assist in selecting the most appropriate filter fabric for the project.

Topic 652 - Subsurface Drainage and Storm Water Management

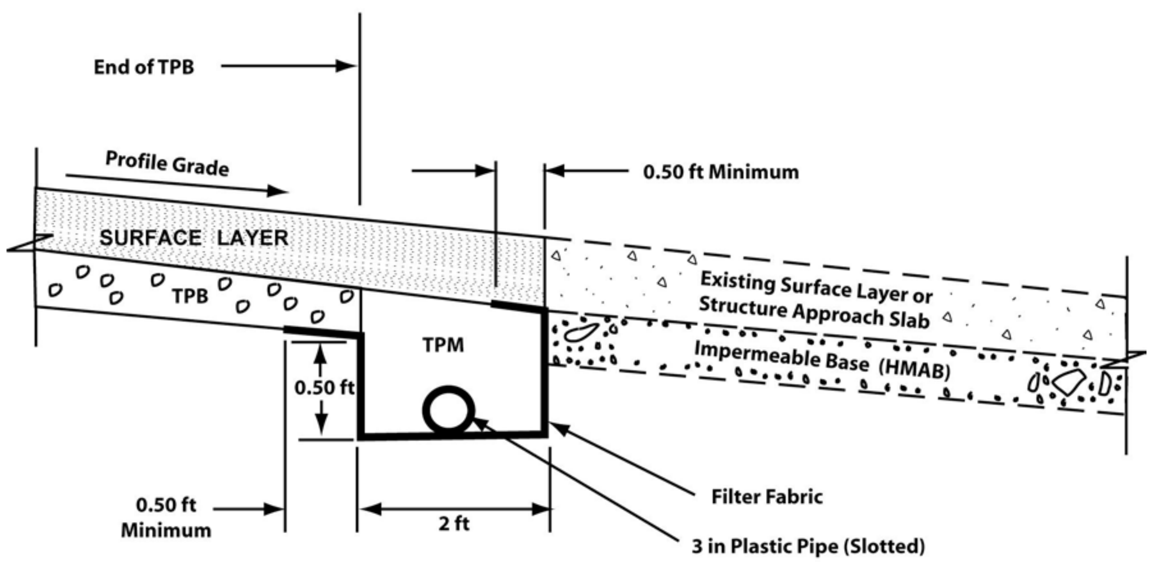
Subsurface drainage (edge drains and underdrains) is to be handled in accordance with the procedures provided in Chapter 890 of the HDM for conveyance and with the procedures in the Storm Water Quality Handbook - Project Planning and Design Guide (PPDG) for storm water compliance. Storm Water Best Management Practices (BMPs) are to be incorporated in the design of projects as prescribed in the PPDG. The PPDG and other information on storm water management can be

Figure 651.2C

Cross Drain Interceptor Trenches



Intermediate Cross Drain
(Longitudinal Section)



Terminal Cross Drain
(Longitudinal Section)

found at Storm Water page of the Division of Design website.

Topic 653 – Other Considerations

653.1 New Construction Projects

The surface layer should employ materials that will minimize surface water intrusion and any joints should be sealed. If sufficient right of way is available, it is preferable to grade the roadbed to allow for a free draining outlet for the pavement rather than installing edge drain. When a free drainage outlet is used, the TPB and AB layers of the pavement must be daylighted on the low end of the section.

On curvilinear alignments, superelevation of the roadway may create depressions at the low side of pavement where the collected water cannot be drained away. An adjustment to the profile grade may be necessary to eliminate these depressions. Refer to Chapter 200 for superelevation guidelines.

653.2 Widening Projects

The widened pavement layers should be engineered to discharge any existing water collected by the pavement. This may be done by extending any drainage layer of the existing adjacent pavement while still providing sufficient pavement structure to meet the pavement design life requirements in Topic 612. The widened layers should extend the full width of the roadbed to a free outlet, if feasible, as in new construction (See Figure 651.2A).

653.3 Rehabilitation and Reconstruction Projects

The surface of the traveled way and shoulders should employ methods and materials that will help minimize surface water intrusion and any joints should be sealed. Saturation or soft spots should be identified and drainage system should be incorporated to restore or repair the existing pavement, if applicable.

653.4 Ramps

Provisions for positive, rapid drainage of the structural section is very important on ramps as much as main lanes. However, including drainage systems in ramp pavements can sometimes create drainage problems such as accumulation of water in the subgrade of descending ramps approaching local street intersections in flat terrain. Such situations, where there may be no cost effective way to provide positive drainage outlets, call for careful evaluation of local conditions and judgment in determining whether a drainage system should be included or not in each ramp pavement structure.

653.5 Roadside Facilities

The surface of parking areas should be crowned or sloped to minimize the amount of surface water penetrating into the pavement. Drainage facilities for the surface runoff should be provided if flexible pavement is used. A mix using ⅜ inch or ½ inch maximum aggregate is recommended to provide a relatively low permeability. The flexible pavement should be placed in one lift to provide maximum density.

CHAPTER 660
PAVEMENT FOUNDATIONS

Topic 661 - Engineering Considerations

Index 661.1 – Description

Pavement foundations typically consist of the following pavement structure layers:

- Base;
- Subbase including stabilized soils;
- Subgrade or basement soil.

Depending on the type of pavement project and other design considerations, a pavement structure may or may not include base, subbase, or both base and subbase layers. The subbase generally consists of lower quality materials than the base, but better than the subgrade or basement soils. When needed, pavement foundation materials are treated to improve strength. The most common treatment materials are cement, asphalt, and geosynthetics.

661.2 Purpose

Pavement foundations serve as a support for the surface layer and distribute the wheel load to subgrade material.

In addition to functioning as part of the pavement structure, bases and subbases serve the following functions:

- Slow down the intrusion of fines from the subgrade soil into pavement structural layers.
- Minimize the damage of frost action.
- Prevent the accumulation of free water within or below the pavement structure.
- Provide a working platform for construction equipment.

Topic 662 - Types of Bases

662.1 Aggregate Base

Aggregate bases consist of a combination of sand, gravel, crushed stone and recycled material. They are classified in accordance with their gradation and the amount of fines. The gradation of the aggregates can affect structural capacity, drainage, and frost susceptibility. The quality of aggregate base material affects the rate of load distribution and drainage.

662.2 Treated Base

(1) *Hot Mix Asphalt Base (HMAB).* Depending on the quality of aggregate, HMAB is classified as dense graded Type A or Type B Hot Mix Asphalt, (HMA). Type A is primarily a crushed aggregate, which provides greater stability than Type B. When used with HMA pavement, the HMAB is to be considered as part of the pavement layer. The HMAB will be assigned the same gravel factor, G_f, as the remainder of the HMA in the pavement structure.

(2) *Concrete Bases.* Concrete base (CB) and Lean concrete base (LCB) are plant-mixed concrete products used as base. CB is essentially unreinforced concrete pavement, constructed with or without reduced joints, used primarily for widening rigid pavement structures that have been or will be surfaced with HMA. CB is finished in anticipation of being paved with HMA. LCB is produced with less cementitious material and allows lower quality aggregates than CB. LCB is primarily intended for concrete pavement structures. Concrete bases can utilize materials that develop strength and/or set quickly. Rapid strength concrete base (RSCB) and lean concrete base rapid setting (LCBRS) have the same applications as CB and LCB, but are usually specified for projects with short construction windows such as individual slab replacement.

(3) *Treated Bases.* Treated bases are granular materials mixed with asphalt or portland cement to improve the strength or stiffness. Treated bases include cement treated base (CTB) and asphalt treated base (ATB). CTB has shown poor performance under rigid pavement in the past. CTB exhibits excessive pumping, faulting, and cracking. This is most likely due to impervious nature of the base, which traps moisture and yet can break down and contribute to the movement of fines beneath the slab.

662.3 Treated Permeable Base

Treated permeable bases (TPB) provide a strong, highly permeable drainage layer within the pavement structure. The binder material may be either asphalt (ATPB) or portland cement (CTPB). Either of these TPB layers will generally provide greater drainage capacity than is needed. The standard thickness is based primarily on constructability with an added allowance to compensate for construction tolerances. If material other than ATPB and CTPB with a different permeability is used, it is necessary to check the permeability and adequacy of the layer thickness. TPB must be used in accordance with a positive sub-drainage system per Index 651.2.

Erosion and stripping (water washing away cement paste, binders, and fines) can be an issue for TPB. Research conducted in the 1990s at the University of California Pavement Research Center (UCPRC) indicates that the use of ATPB is highly susceptible to stripping. Because of this, the Department recommends use of standard aggregate base (AB), with a compaction of the HMA layer of at least 93 percent of theoretical Rice maximum, instead of ATPB for new pavement structures. When ATPB is needed, such as to ensure continuity of existing ATPB/CTPB layer and/or provide drainage through the pavement structure, special provisions should be made to ensure that it is not subjected to conditions that will lead to premature structural failure. The following guidelines should be followed when using ATPB on State highway pavement projects.

(1) *Considerations for using ATPB.* The following two conditions warrant consideration to use ATPB layer in the pavement structure:

   (a) When widening or adding lanes adjacent to an existing ATPB layer to ensure continuity of existing ATPB layer.

   (b) Where there is need to drain excess water through the pavement, such as when the uphill side of pavement does not allow for drainage. However, when practical, it is better in such cases to use sub-surface drainage to carry water to the other side of the roadway rather than drain excess water through an ATPB layer just below the HMA.

(2) *Added features when using ATPB.* The following features are recommended when using ATPB:

   (a) Use edge drains or daylight the edges (see Figure 651.2A in Chapter 650).

   (b) If using edge drains, be sure that Maintenance is informed and can budget funds for maintaining edge drains. Developing an estimate of maintenance costs to maintain edge drains and Budget Change Proposals may be required to assure edge drains can be maintained.

   (c) Try to use permeable backfill in shoulders on sides of edge drain to avoid bathtub effect if edge drain becomes clogged.

   (d) Increase binder content to 3 percent (maybe higher)

   (e) Tack coat each layer.

   (f) Perform moisture sensitivity testing on ATPB.

   (g) Compaction of the HMA layer should be at least 93 percent of theoretical Rice maximum.

662.4 Subbase

Aggregate subbase is similar to aggregate base but with less restrictive quality requirements. Because of continual depletion of quarry aggregates, most subbases typically consist of recycled pavement materials or quarry products than cannot meet the criteria for aggregate base.

Excavated soil and low quality imported borrow material can be chemically treated with a cementitious binder to improve strength and reduce expansiveness. The most common types of stabilized soils are lime stabilized soil (LSS) and cement stabilized soil (CSS). Other soil stabilization agents include asphalt and fly ash or kiln dust, but these are considered experimental alternatives and are not currently supported in the Department's Standard Specifications or guidelines.

Stabilizing the soil does not eliminate or reduce the required aggregate subbase for rigid or composite pavements in the rigid pavements catalog (see Topic 623). However, for flexible pavements, stabilized soil can be used as a substitute for all or part of the required subbase.

The District Materials Engineer should be contacted to assist with the selection of the most appropriate method to stabilize soils for individual projects. Final decision as to which stabilization to use rests with the District.

Topic 663 – Engineering Properties for Base and Subbase Materials

663.1 Selection Criteria

Because different types of treated and untreated base or subbase materials have different capacities for resisting forces imposed by traffic loads, this factor must be considered when determining the thickness of pavement elements. Besides load carrying consideration, the final selection of the bases or subbases for a given project depends on several other factors such as available materials, terrain, climate, economics, and past performance of the pavement under similar project or climate conditions and travel patterns. The District Materials Engineer should be contacted for the latest guidance in base and subbase materials among other related engineering considerations.

663.2 Base and Subbase for Rigid Pavements

For rigid pavements, the capacity of base and subbase materials to resist traffic loads is considered in the design catalogs found in Topic 623. Table 663.2 provides the properties for base and subbase materials used for the Rigid Pavements design catalogs.

663.3 Base and Subbase for Flexible Pavements

For flexible pavements, the capacity of treated or untreated base and subbase materials to resist traffic loads is considered by use of the California R-value and the gravel factor, G_f, which expresses the relative stiffness of various materials when compared to gravel. Table 663.3 provides the California R-value and G_f for base and subbase materials used in flexible pavements.

When the stabilized soil is substituted for aggregate subbase for flexible pavements, as discussed in Index 663.4, the actual thickness of the stabilized soil layer is obtained by dividing the GE by the appropriate G_f. The G_f is determined based on unconfined compressive strength (UCS) of the stabilized material as follows:

$$G_f = 0.9 + \frac{UCS(psi)}{1000}$$

This equation is only valid for UCS of 300 psi or higher at 28 days cure. For cement and lime stabilization, UCS is determined by different test methods, but in both cases the 28-day UCS is simulated by curing prepared samples in an oven for 7 days. The gravel factor G_f allowed using this equation should range from a minimum of 1.2 to a maximum of 1.7.

Because the stabilization of soil may be less expensive than the base material, the calculated base thickness can be reduced and the stabilized soil thickness increased. The base thickness is reduced by the corresponding gravel equivalency provided by the cement or lime stabilized soil. The maximum thickness of lime treated subgrade is limited to 2 feet.

Topic 664 - Subgrade Enhancement

664.1 Overview

Properties of low quality subgrade can be improved to provide a platform for the construction of subsequent layers and to provide adequate support for the pavement over its design life. The most common methods used in the Department for subgrade improvement include:

- Mechanical stabilization;
- Chemical stabilization; or
- Subgrade enhancement geosynthetics, see Topic 665.

664.2 Mechanical Subgrade Enhancement

Improving strength is usually the primary reason for implementing mechanical stabilization. Mechanical subgrade enhancement includes the following:

(1) *Compaction*. Sufficient strength can often be achieved on certain subgrade materials that do not quite meet the design requirements by

Table 663.2

Base and Subbase Material Properties for Rigid Pavement Catalog

HMA Type A Properties	
Aggregate gradation	0% retained on ¾ inch sieve 32% retained on ⅜ inch sieve 52% retained on No. 4 sieve 5.5% passing No. 200 sieve
Asphalt binder type	See Index 632.1(2) and Table 632.1
Reference temperature	70 °F
Poisson's ratio	0.35
Effective binder content	11.662%
Air voids	8%
Total unit weight	149 lb/ft^3
Thermal conductivity	0.657 Btu/hr ft °F
Heat capacity	0.23 Btu/lbm-°F
Base erodibility index (1)	2
LCB / LCBRS(1) Properties	
Unit weight	150 lb/ft^3
Poisson's ratio	0.20
Elastic modulus	2.00 x 10^6 psi
Thermal conductivity	15 Btu-in/h-ft^2-°F
Heat capacity	0.28 Btu/lbm-°F
Base erodibility index (1)	1
AB / AS Properties	
Poisson's ratio	0.40
Coefficient of lateral pressure, K_0	0.5
Resilient modulus for AB	43,500 psi
Resilient modulus for AS	29,000 psi
Plasticity Index	1
Passing No. 200	3%
Passing No. 4	20%
D_{60}(2)	0.315 inch
Base erodibility index(3)	4

NOTES:

(1) LCB / LCBRS = Lean Concrete Base / Lean Concrete Base Rapid Setting

(2) D60 = Particle diameter at which 60 percent of the material sample is finer than or would pass a sieve size of that diameter.

(3) Base erodibility index is classified as a number from 1 to 5 as follows:

1 = Extremely erosion resistant material

2 = Very erosion resistant material

3 = Erosion resistant material

4 = Fairly erodible material

5 = Very erodible material

Table 663.3
Gravel Factor and California R-values for Bases and Subbases

Type of Material	Abbreviation	California R-value	Gravel Factor (G_f)
Aggregate Subbase	AS-Class 1	60	1.0
	AS-Class 2	50	1.0
	AS-Class 3	40	1.0
	AS-Class 4	specify	1.0
	AS-Class 5	specify	1.0
Aggregate Base	AB-Class 2	78	1.1
	AB-Class 3	specify	1.1(1)
Asphalt Treated Permeable Base	ATPB	NA	1.4
Cement Treated Base	CTB-Class A	NA	1.7
	CTB-Class B	80	1.2
Cement Treated Permeable Base	CTPB	NA	1.7
Lean Concrete Base	LCB	NA	1.9
Lean Concrete Base Rapid Setting	LCBRS	NA	1.9
Hot Mix Asphalt Base	HMAB	NA	(2)
Lime Stabilized Soil	LSS	NA	0.9+(UCS/1,000)
Cement Stabilized Soil	CSS	NA	0.9+(UCS/1,000)

NOTES:

(1) Must conform to the quality requirements of AB-Class 2.

(2) When used with HMA, the HMAB is to be considered as part of the pavement layer. The HMAB will be assigned the same G_f as the remainder of the HMA in the pavement structure.

Legend:

NA = Not Applicable

UCS = Unconfined Compressive Strength in psi (minimum 300 psi per California Test 373) for lime and ASTM D 1633 (modified) for cement) additional compaction usually with a heavier or different type of roller than is normally used. Compaction improves aggregate interlock, and reduces air-void content, pore connectivity, and consequent susceptibility to moisture ingress.

(2) *Blending*. Blending involves the mixing of materials that have different properties (typically particle size distribution) to form a material with characteristics that improve upon the limitations of the source materials. In most instances, blending will involve adding coarse aggregates to the finer in situ material. Less common in California is the addition of fine material to in situ sandy or coarse aggregates to fill voids and obtain a denser gradation.

664.3 Chemical Stabilization

Low quality in situ subgrade soil can be improved from Type III to Type II or Type I (see Table 623.1A) by chemical stabilization to a minimum depth of 0.65 foot using an approved stabilizing agent such as lime, cement, asphalt, or fly ash (asphalt and fly ash are not currently supported in the Department's Standard Specifications or guidelines). Chemically treated soil samples should be tested to determine the unconfined strength for the stabilized soil. To ensure long-term stability of the subgrade during the pavement design life the stabilized soil should achieve an initial minimum unconfined strength of 300 psi.

664.4 Subgrade Enhancement Geosynthetics

Subgrade enhancement geosynthetics are fabrics or grid interlayers placed between the pavement structure and the subgrade (the subgrade is usually untreated). Geosynthetics can be used for temporary improvement of subgrade to provide a platform for equipment during construction, and/or long-term enhancement to improve the ability to sustain traffic loads distributed to the subgrade. Detailed information on subgrade enhancement geosynthetics is provided in Topic 665.

Topic 665 - Subgrade Enhancement Geosynthetic Fabrics

665.1 Purpose

Subgrade Enhancement Geosynthetic (SEG) can be either a Subgrade Enhancement Geotextile (SEG_T) or Subgrade Enhancement Geogrid (SEG_G) placed between the pavement structure and the subgrade (the subgrade is usually untreated). The placement of SEG below the pavement will provide subgrade enhancement by bridging soft areas and providing a separation between soft subgrade fines susceptible to pumping and high quality subbase or base materials. On weak subgrade, the use of SEG can provide stabilization (the coincident function of separation and reinforcement). As the soft soil undergoes deformation, properly placed SEG when stretched will develop tensile stress. Other benefits of using SEG include:

- Prevent premature failure and reduce long-term maintenance costs;
- Potential cost savings:
  - o Reduce subbase or aggregate base thickness in some situations,
  - o Reduce or eliminate the amount of soft or unsuitable subgrade materials to be removed,
- Increased performance life and reliability of the pavement;
- Prevent contamination of the base materials;
- Better performance of a pavement over soils subject to freeze/thaw cycles;
- Reduced disturbance of soft or sensitive subgrade during construction; and
- Ability to install in a wide range of weather conditions.

665.2 Properties of Geosynthetics

(1) *Subgrade Enhancement Geotextile (SEG_T)*. Mechanical, physical, and other properties of geotextile (SEG_T) used for subgrade enhancement shall meet the requirements in Section 88 of the Standard Specifications.

(2) *Subgrade Enhancement Geogrid (SEG_G).* Property requirements for SEG_G are related to performance. The most important geogrid properties for subgrade enhancement related to performance and durability are tensile strength, junction strength, flexural rigidity, and aperture size.

Different types of geogrid can be used for SEG_G provided their stabilizing performance is equivalent to or greater than the values specified in Section 88 of the Standard Specifications.

665.3 Required Tests

The following geotechnical soil laboratory tests are required to evaluate subgrade for the geosynthetic applications:

- Atterberg Limit Tests: CT 204 or alternatively ASTM D 4318 or AASHTO T90
- R-value test: CT 301 or alternatively California Bearing Ratio (CBR) test (ASTM D1883) or AASHTO T 193
- Sieve Analysis: CT 202 or alternatively ASTM C136 or AASHTO T27

665.4 Mechanical Stabilization Using SEG_T and SEG_G

SEG_T and SEG_G achieve mechanical stabilization through slightly different mechanisms:

(1) *Subgrade Enhancement Geotextile (SEG_T).* A geotextile's primary stabilization mechanism is filtration and separation of a soft subgrade and the subbase or base materials. The sheet-like structure provides a physical barrier between these materials to prevent the aggregate and subgrade from mixing. It can also reduce excess pore water pressure through a mechanism of filtration and drainage. Secondary mechanisms of a geotextile are lateral restraint and reinforcement. Lateral restraint is achieved through friction between the surface of the geotextile and the subbase or base materials. Reinforcement mechanism requires deformation of the subgrade and stretching of the geotextile to engage the tensile strength and create a "tensioned membrane."

(2) *Subgrade Enhancement Geogrid (SEG_G).* The primary stabilization mechanism of a geogrid is lateral restraint of the subbase or base materials through a process of interlocking the aggregate and the apertures of the geogrid. The level of lateral restraint that is achieved is a function of the type of geogrid and the quality and gradation of the base or subbase material placed on the geogrid. To maximize performance of the geogrid, a well-graded granular base or subbase material should be selected that is sized appropriately for the aperture size of the geogrid. When aggregate is placed over geogrid, it quickly becomes confined within the apertures and is restrained from punching into the soft subgrade and shoving laterally. This results in a "stiffened" aggregate platform over the geogrid. Very little deformation of the geogrid is needed to achieve the lateral restraint and reinforcement. Separation and filtration/vertical drainage are secondary mechanisms of a geogrid. Because the aggregate is confined within the apertures of the geogrid and cannot move under load, separation and filtration can be achieved.

665.5 Selecting Geosynthetic Type and Design Parameters

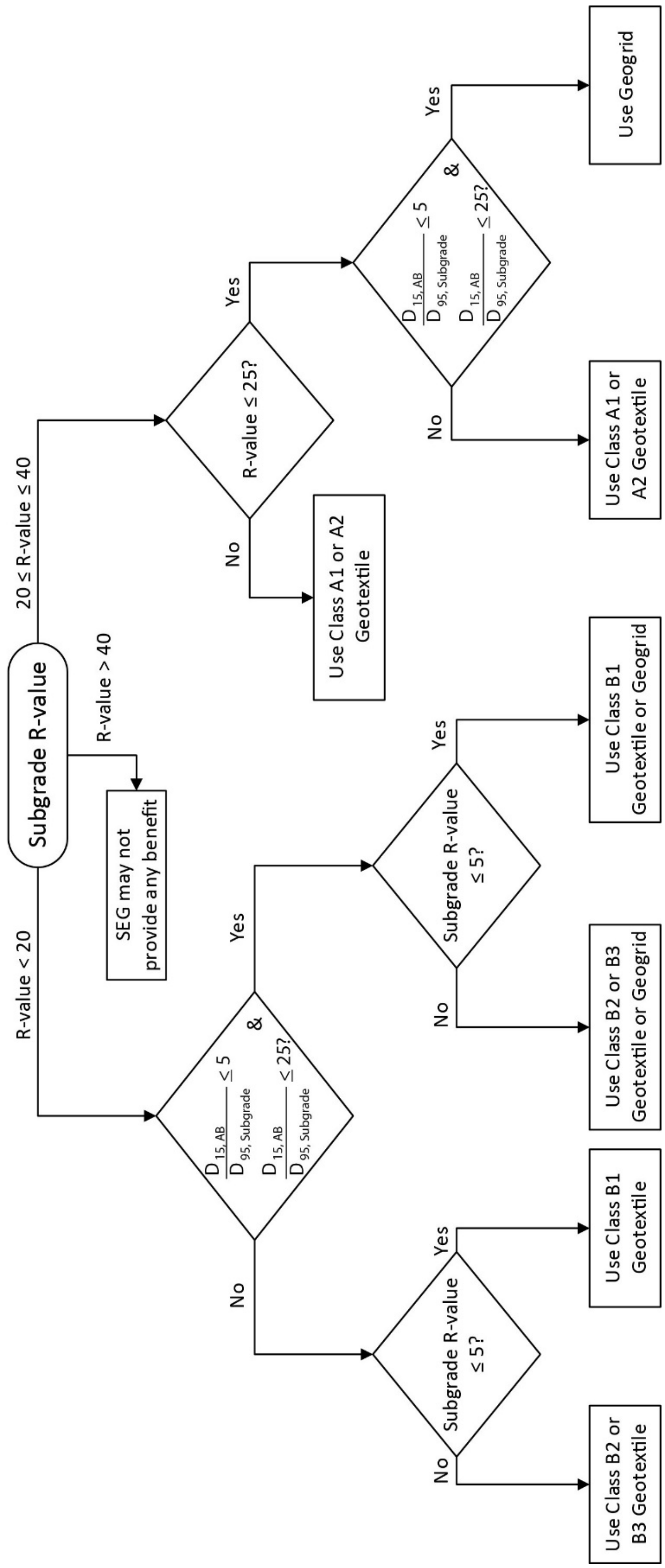
(1) *Determining SEG Functions* - Subgrade stabilization is the primary function for geogrids installed between an aggregate base and subgrade layer. The primary functions of geotextiles are separation, stabilization, filtration, reinforcement, and drainage. Figure 665.5 shows a flowchart to aid in selecting the possible functions and types of geotextile and geogrid.

(2) *Selecting SEG* - SEG can be selected based on the following criteria:

- For subgrade R-values greater than 25 but less than 40, SEG_T is recommended to use for its separator function. The requirements for separator function can be found in Section 88 of the Standard Specifications.
- For subgrade R-value between 20 and 25, SEG_G should be used for its stabilization function. The stabilization requirements for SEG_G can be found in Section 88 of the Standard Specifications.

Figure 665.5

Flowchart for Selecting an Appropriate SEG



- For subgrade R-value less than 20, a designer should choose either SEG_G or SEG_T.

Before selecting SEG, the engineer should investigate the potential engineering and economic benefits of using SEG_T or SEG_G.

665.6 Application of SEG

(1) *Appropriate Applications* - Locations that may require placement of SEG include areas with the following soil characteristics:

- Poor (low strength) soils, which are classified in the Unified Soil Classification System (USCS) as clayey sand (SC), lean clay (CL), silty clay (ML-CL), high plastic clay (CH), silt (ML), high plasticity or micaceous silt (MH), organic soil (OL/OH), and peat (PT);
- Low undrained shear strength: C_u < 2,000 psf, and/or other properties stated below in Index 665.6(2);
- High water table and high soil sensitivity
- Shallow utilities or contaminated soils.

(2) *Conditions for Using SEG:*

- SEG_G is most applicable for R-values < 25 or CBR < 3.5 or M_R < 5,000 psi. For R-value between 25 and 40 or CBR between 3.5 and 6.5 or M_R between 5,000 and 9,500 psi the engineer may consider utilizing a geogrid for base reinforcement.
- SEG_T is most applicable for R-value < 20 or CBR < 3 or M_R < 4,500 psi. For R-values between 20 and 40 or CBR between 3 and 6.5 or M_R between 4,500 and 9,500 psi, the engineer may consider utilizing a SEG_T as a separator.
- On very soft subgrade conditions (R-value < 10 or CBR <2 or M_R < 3,000 psi), consider placing a thicker initial lift (minimum of 6 inches) of subbase or aggregate base material on top of SEG to effectively bridge the soft soils and avoid bearing capacity failure under construction traffic loading.
- Use of geogrid is not recommended unless the aggregate material meets the following natural filter criteria:
  - D_{15}Aggregate Base / D_{85}Subgrade ≤ 5 and D_{50}Aggregate Base / D_{50}Subgrade ≤ 25.
  - D_{15}, D_{85}, and D_{50} are grain sizes of the soil particles for which 15 percent, 85 percent, and 50 percent of the material is smaller than these sieve sizes.
- If the aggregate base material does not meet the above natural filter criteria, geotextiles that meet both separation and stabilization requirements are recommended.
- Do not use geosynthetics for subgrade with R-value > 40 or CBR > 6500 or M_R > 9,500 psi, because stabilization of subgrade is not required and application of geosynthetics will not impart significant benefit to the pavement.

665.7 Other Design Considerations

The following should also be considered by the design engineer when designing pavements involving SEG:

- On soft subgrade soils, the SEG may replace some or all stabilizing material such as lime or cement used solely as a working platform to provide access to construction equipment.
- For information on how to mitigate for expansive subgrade consisting of clay soils with plasticity index greater than 12, see Index 614.4.
- Consider using SEG for longer life pavement, if not otherwise specified.
- Perform a filter analysis if the soil material types described in Index 665.6(1) are either above or below SEG_G to determine whether natural filter criteria are met to control migration of fines into the subbase or aggregate base materials.
- For applications involving drainage and filtration, the design engineer should verify that

the permeability of the SEG_T is greater than the permeability of the soil.

- If a SEG_T is to be placed in direct contact with recycled concrete material, SEG_T made of polyester should not be used. Otherwise, a separating layer of thickness greater than 0.3 feet must separate the geotextile from the recycled concrete material.
- SEG is not necessary if the subgrade is planned for chemical stabilization such as lime or cement treatment.

665.8 R-value Enhancement Using SEG

Subgrade soils with R-value <20 are considered poor or weak soils and require SEG to provide reinforcement as the primary function and separation as the secondary function. However, depending on type and treatment of the base layer, pavements constructed over subgrade soils with R-value up to 40 can benefit from separation if the subgrade soil contains an appreciable amount of fines. The SEG when placed with aggregate subbase provides a working platform for access of construction equipment, mainly on subgrade with R-values of 5 to 10.

The use of SEG on weak subgrade (with R-value <20) can increase the effective R-value of such soils. Therefore, the benefit of using SEG on such weak soils can be realized though using thinner aggregate bases or subbases in flexible pavement design. Likewise, SEG can also affect the design of rigid pavements by providing a stronger subgrade system.

The following R-values are recommended when SEG is used on subgrade with low R-value less than 25:

- For subgrade with an R-value of less than 20, a design R-value of 20 can be used if SEG_T is utilized.
- When subgrade has an R-value of less than 25, a design R-value of 25 can be used if SEG_G is utilized. Additional geotextile separator may be used unless the aggregate base material meets the natural filter criteria presented in Index 665.6(2).

Additional information on the use of SEG and the selection of the appropriate properties of the SEG based on project specifics are explained in the "Subgrade Enhancement Geosynthetic Design and Construction Guide" on the Department Pavement website.

665.9 SEG Abbreviations and Definitions

The following is a list of definitions related to subgrade enhancement geosynthetics and their applications:

Apparent Opening Size: A geotextile property that indicates the approximate diameter of the largest soil particle that would effectively pass through the geotextile. Commonly, 95 percent of the geotextile openings are required to have that diameter or smaller as measured using ASTM D 4751.

Aperture Shape: Describes the shape of the geogrid opening.

Aperture Size: Dimension of the geogrid opening.

D_{15}: The particle (or grain) size represented by the "15 percent passing" point when conducting a sieve analysis of a soil sample.

D_{50}: The particle (or grain) size represented by the "50 percent passing" point when conducting a sieve analysis of a soil sample.

D_{85}: The particle (or grain) size represented by the "85 percent passing" point when conducting a sieve analysis of a soil sample.

Filtration: The process of allowing water out (perpendicular to plane of geotextile) of a soil mass while retaining the soil.

Geogrid: A geosynthetic formed by a regular network of integrally connected tensile elements with apertures of sufficient size to allow "strike-through" and interlocking with surrounding soil, rock, or earth to improve the performance of the soil structure.

Geosynthetic: A group of synthetic materials made from polymers that are used in many transportation and geotechnical engineering applications. .

Geotextile: A permeable sheet-like geosynthetic which, when used in association with soil, has the ability to provide the functions of separation, filtration, reinforcement, and drainage to improve the performance of the soil structure.

Grab Tensile Strength: The maximum force applied parallel to the major axis of a geotextile test specimen of specified dimensions that is needed to tear that specimen using ASTM D 4632.

Nonwoven Geotextile: A planar geotextile typically manufactured by putting small fibers together in the form of a sheet or web, and then binding them by mechanical, chemical and/or solvent means.

Permeability: The permeability of soil or geotextile is the flow rate of water through a soil or geotextile. The permeability of geotextile can be determined by permittivity, which can be measured using ASTM D 4491, multiplied by its effective thickness and the permeability of soil can be measured using ASTM D 2434 or 5084.

Permittivity: It is the volumetric flow rate of water per unit cross-section area of a geotextile, per unit head, in the normal direction through a material as measured using ASTM D 4491.

Puncture Strength: The measure of a geotextile's resistance to puncture determined by forcing a probe through the geotextile at a fixed rate using ASTM D 6241. 10

Reinforcement: The improvement of the soil system by introducing a geosynthetic to enhance lateral restraint, bearing capacity, and/or membrane support.

Resilient Modulus: The ratio of applied deviator stress to recoverable or resilient strain.

Separation: A geotextile function that prevents the intermixing between two adjacent dissimilar materials, so that the integrity of the materials on both sides of the geotextile remains intact.

Stabilization: The long-term modification of the soil by the coincident functions of separation, filtration, and reinforcement furnished by a geosynthetic.

Tear Strength: The maximum force required to start or to propagate a tear in a geotextile specimen of specified dimensions using ASTM D 4533.

Ultraviolet Stability: The ability of a geosynthetic to resist deterioration from exposure to the ultraviolet rays of the sun as tested using ASTM D 4355.

Woven Geotextile: Produced by interlacing two or more sets of yarns, fibers, or filaments where they pass each other at right angles.

CHAPTER 670 TAPERS AND SHOULDER BACKING

Topic 671 - Pavement Tapers

Index 671.1 Background and Purpose

Pavement tapers are a common design detail for asphalt layer overlays and other projects where new pavement surface has a higher profile than existing pavement surface or curbs. The goal of tapers is to provide a smooth unnoticeable transition from pavement to pavement. Tapers are intended to provide a reasonable cost alternative to engineering a profile for every transition. However, in some cases, an engineered profile may be more cost-effective than a taper.

This section provides information on the best design practices for transition tapers that meet geometric, operational, constructability, as well as other pavement surface and drainage standard practices. The tapers presented in this index meet the Caltrans standards and requirements for grade breaks in Index 204.4. The pavement tapers discussed herein do not address every possible situation that can be encountered on projects throughout the State. Good engineering judgment should still be exercised when developing transition taper details for a specific project. This index only addresses permanent pavement transition tapers used on overlay and other pavement projects.

671.2 Engineering Requirements and Considerations

(1) *Minimum Thickness Requirement.* In order for tapers to be constructable, maintainable and meet performance requirements:

(a) The minimum thickness for an asphalt pavement taper should be no less than 3 times the maximum aggregate size (example 0.15’ for ½” aggregate and 0.20’ for ¾” aggregate.)

(b) The minimum thickness of the overall surface course layer (existing and new) in the taper should be no less than that of the adjoining existing pavement.

(c) When tapering into an existing pavement that was previously overlaid (pavement preservation or rehabilitation), the new taper should overlap the taper of the previous overlay to avoid creating a “dip” or “weak spot” in the pavement (see Figure 671.2A).

(2) *Transition Taper Slopes.* The taper slope should be 200:1 or flatter, with taper slope of 400:1 being preferred in highways with design speeds of 65 mph or higher. At locations where taper slopes flatter than 400:1 are desired, engineered profiles should be used because they are often shorter, less expensive, and easier to construct than the pavement taper.

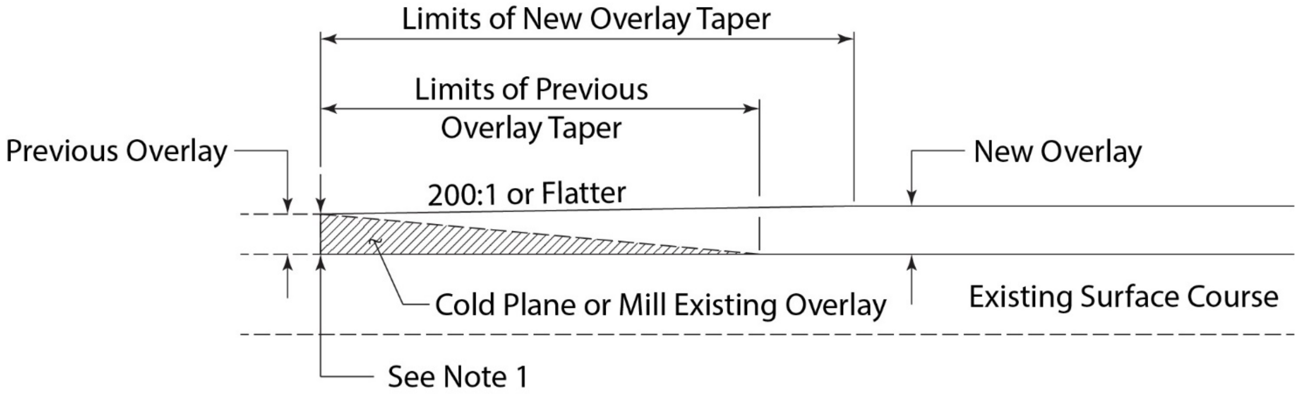
(3) *Design Life Requirements for Tapers.* For new construction, widening, and rehabilitation/reconstruction projects, the minimum thickness of the pavement structure (existing plus surface course overlay) for pavement tapers must meet the minimum pavement design life requirements for the project as discussed in Topic 612. This is intended to prevent creating isolated “weak spots” in the pavement that may require additional maintenance and repair in the future. On rehabilitation and reconstruction projects, where the pavement structure of the taper does not meet the pavement design life requirements, the pavement structure or part of it will need to be removed and replaced. Deviations from this requirement or decision not to reconstruct the pavement sections underneath bridges will require a mandatory design exception from Headquarters Pavement Program for pavement design life (see Index 612.2 and 612.5). Since pavement preservation projects (preventive maintenance and CAPM projects) are not designed for structural capacity, the minimum thickness of the pavement structure for the pavement taper needs only to match or exceed the existing pavement structure. See Figure 671.2B for further details.

671.3 Tapers into Existing Pavement or Structure

Figures 671.3A to 671.3C provide details on how to construct pavement tapers.

Figure 671.2A

Tapering Into a Previously Overlaid Pavement



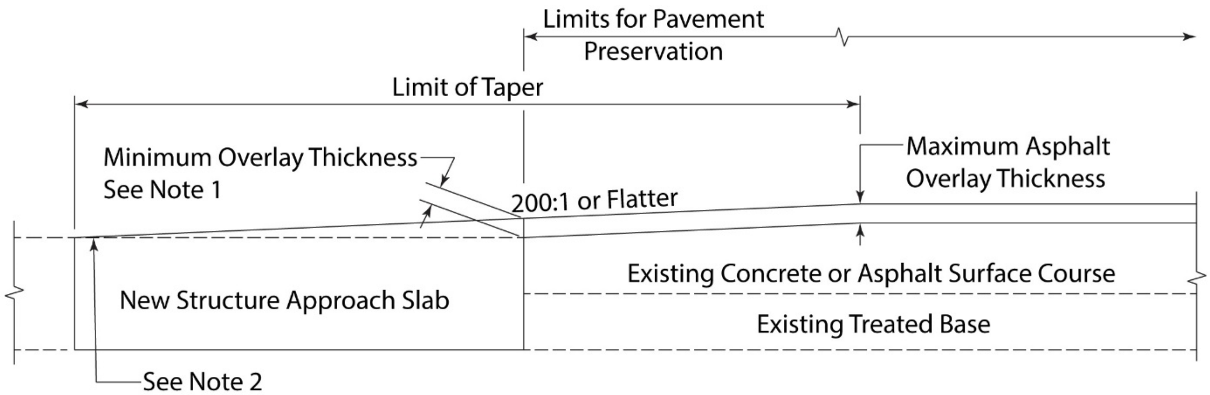
NOTES:

(1) Minimum thickness should match thickness of previous overlay.

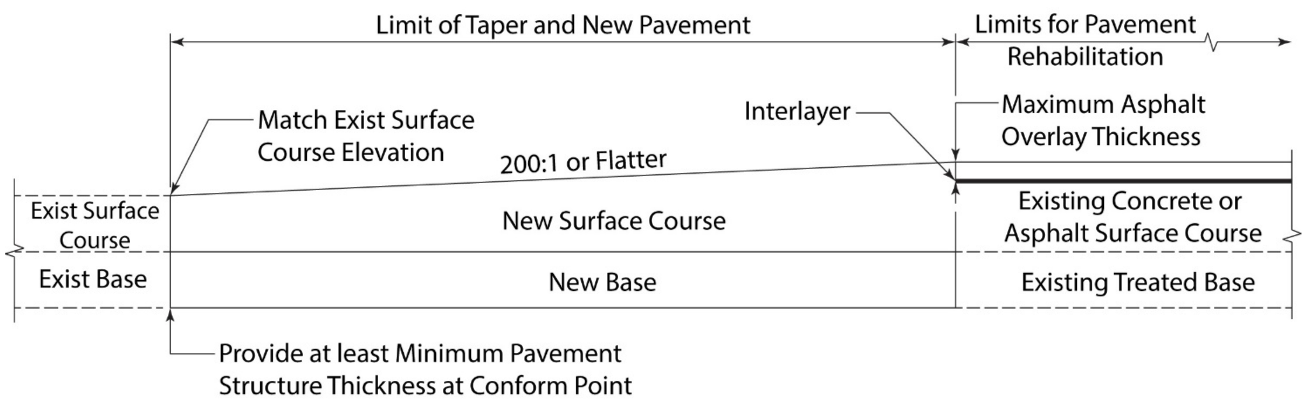
(2) No Scale.

Figure 671.2B

New Structure Approach Pavement Transition Details



Pavement Preservation



Pavement Rehabilitation

NOTES:

(1) Use Maximum Overlay Thickness or 3x maximum aggregate size, whichever is less.

(2) Cold plane as needed to conform overlay with existing pavement.

(3) No Scale.

(1) *Tapers into an Existing Asphalt Pavement.* Where a new pavement structure or an overlay is tapering into an existing asphalt pavement that is not part of the project, the following apply:

(a) For preventive maintenance projects (thin asphalt overlays of 0.10' or less), the Design Engineer should follow the taper details in Figure 671.3A.

(b) For CAPM projects, taper the overlay using the same details used for OGFC taper to existing OGFC or HMA pavement surface course (See Figure 671.3A)

(c) For rehabilitation projects, taper the overlay using the taper details shown in Figure 671.3A for HMA taper to existing HMA surface course.

(2) *Tapers into an Existing Concrete Pavement.* Where a new pavement structure or an overlay is tapering into an existing pavement that is not part of the project or into/under a structure, grinding existing concrete pavement to create a taper is not recommended because it shortens the life of the concrete pavement. Because it is not always practical to remove and replace concrete pavement for every overlay, the following guidance should be followed regarding tapers for concrete pavement.

(a) For preventive maintenance projects (thin asphalt overlays of 0.10' or less), the taper should follow the taper details for OGFC overlay over asphalt pavement found in Figure 671.3A or reduce the thickness of overlay to 0.08' at end of taper and roll down edge to minimize raveling. For under structures, existing concrete surface may remain.

(b) For CAPM projects, either taper the overlay down using the same details used for OGFC (See Figure 671.3A) or replace the concrete pavement slab. For under structures, the existing concrete surface may remain but should be repaired and ground or rebuilt as needed in accordance with CAPM strategies for concrete pavement in Index 624.2.

(c) For rehabilitation projects, do not grind the concrete pavement to accommodate a taper. Instead, remove concrete pavement within the taper section and replace with a new pavement structure that will meet the design life requirements for the project as defined in Topic 612.

(d) When grinding concrete pavement, meet the following two conditions:

- Use a diamond grinder, not a planing machine.
- Never grind more than 1 inch or reduce the thickness of the concrete pavement slab to less than 0.65 foot.

If neither of these conditions can be attained with the taper detail, then remove and replace the concrete pavement slabs and the underlying base as needed for the transition taper section to match the existing pavement surface.

(3) *Longitudinal Tapers at Shoulders, Curbs, Dikes, Inlets, and Metal Beam Guard Railing.* Detailed drawings and information on the best design practices for longitudinal tapers at shoulders, curbs, dikes, inlets, and metal beam guard railing (MBGR) are shown in Figure 671.3B.

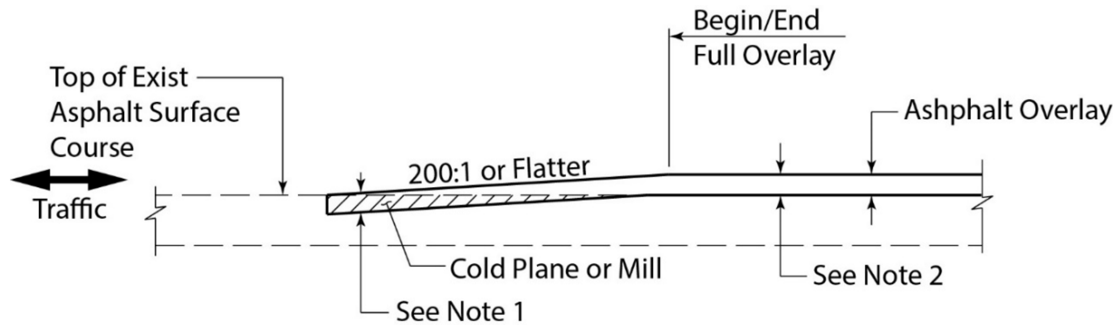
(4) *Tapers Into or Under Structures.* Figure 671.3C provides a layout and information for transition tapers under an existing structure. The following guidance should be followed when designing tapers underneath over-crossings or into bridges:

(a) Compare the cost and constructability of very flat tapers (400:1 or flatter) vs. engineered profiles to ensure that the less expensive and easier to construct alternative is used when replacing pavement underneath a structure.

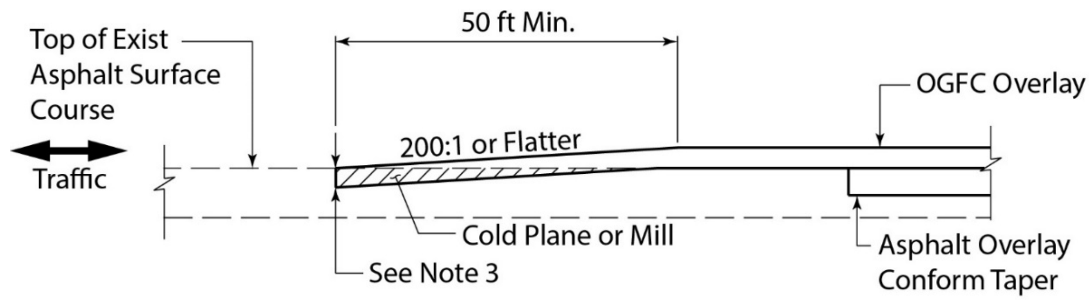
(b) The minimum thickness of the pavement structure for transition tapers into or under bridges must meet the minimum design life requirements discussed in Index 671.2(4) for new construction, widening, rehabilitation, and reconstruction projects.

Figure 671.3A

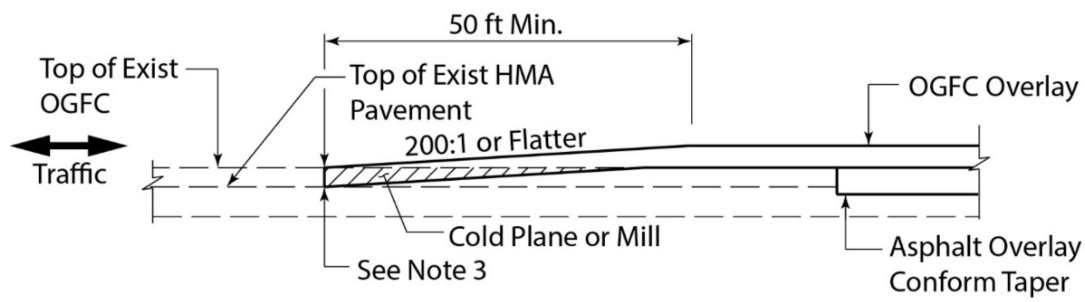
Transverse Transition Tapers for Pavement Preservation Projects



Transverse Asphalt Taper to Existing Asphalt Pavement



Transverse OGFC Taper to Existing Asphalt Pavement



Transverse OGFC Taper to Existing OGFC Pavement

NOTES:

(1) Minimum thickness should match thickness of the top lift.

(2) See HDM for minimum thickness.

(3) Same thickness as OGFC overlay or 0.10’, whichever is less.

(4) Do not use HMA to bring the shoulders up to grade when traveled way is OGFC.

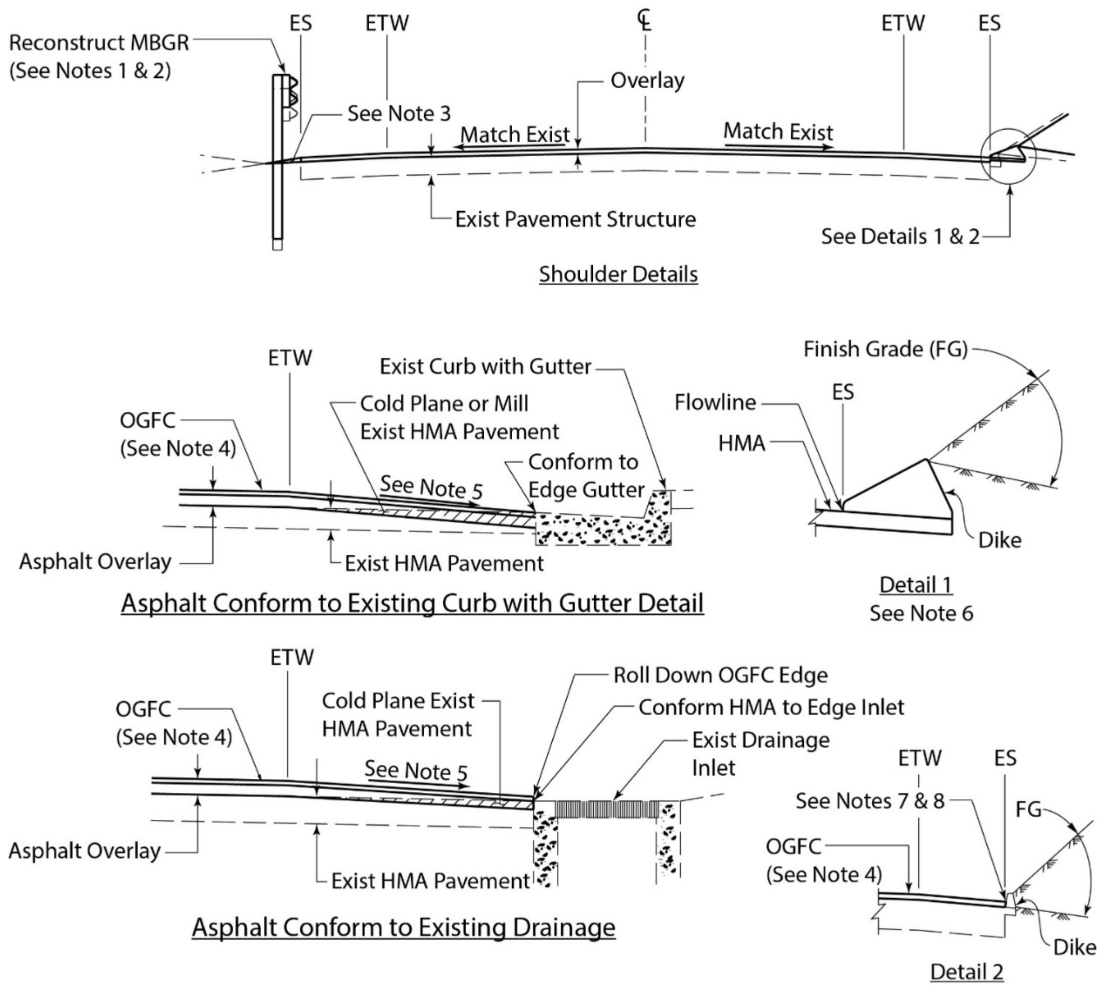
LEGEND:

HMA = Hot Mix Asphalt

OGFC = Open Graded Friction Course

Figure 671.3B

Longitudinal Tapers at Shoulders, Curbs, Dikes, Inlets, and Metal Beam Guard Railing



NOTES:

(1) Additional design and safety criteria may apply for metal beam guard railing (MBGR), for further info, see Traffic Manual or District Traffic.

(2) When grinding or paving next to MBGR or obstacle, reconstructing MBGR will be necessary to accommodate grinding machines and compaction equipment.

(3) Contact District Landscape and Maintenance regarding the appropriate treatment for weed abatement.

(4) OGFC applies only when used as a surface course, omit details for this course when OGFC is not used.

(5) See HDM Topic 302 for maximum allowable cross-slopes.

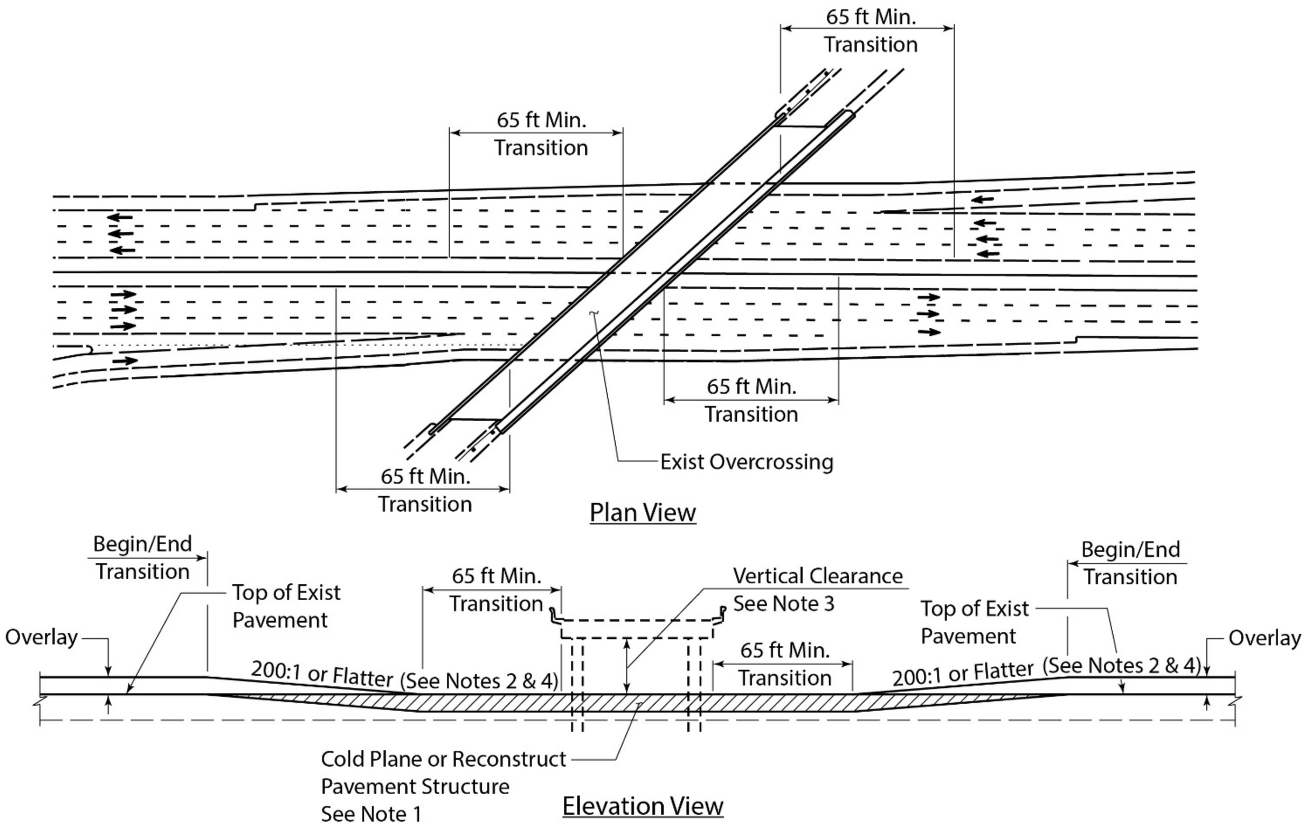
(6) For additional information on dikes, see HDM Topic 303, and Standard Plan A78B.

(7) Verify with Hydraulics to see if dike needs to be raised to maintain capacity of gutter.

(8) Verify with District Hydraulics if additional drainage is required at the conform on the shoulder or at bridge approach slabs in order to avoid ponding.

Figure 671.3C

Transition Taper Underneath Overcrossing/Bridge



NOTES:

(1) Pavement structure thickness needs to provide the proposed pavement design life. This may require that the pavement structure be removed and replaced.

(2) Verify that the existing drainage facilities will continue to function properly after transition is completed.

(3) For minimum vertical clearance requirements, see HDM Index 309.2

(4) Creation of a sag may require additional drainage features.

Topic 672 - Shoulder Backing

672.1 Background and Purpose

(1) *Purpose.* Shoulder backing is a thin course of granular material that is used to provide support to the pavement edge by preventing edge cracking and pavement edge loss. Shoulder backing also minimizes pavement edge drop-off heights for overlays.

(2) *Standards and Requirements.* The placement of shoulder backing requires proper compaction of the shoulder backing material.

(3) *Application*: Shoulder backing is designed to provide edge support for thin overlays placed on existing pavements. Do not use shoulder backing as embankment material in the following cases:

- To repair erosion or subsidence in existing slopes (See Figure 672.3C).
- For side slope reconstruction (See Figure 672.3C).
- In locations where the overlay thickness is greater than 0.50 ft (See Figure 672.3C).
- For backfill behind dikes (See Figure 672.3D).
- To construct the required minimum hinge width (HW) for guardrails, dikes, and barriers.
- In roadside ditches or gutters (See Figure 672.3E). Since the material used for shoulder backing can be erodible, use non-erodible materials or stabilized base material in roadside ditches or gutters.

Shoulder backing is not be used in the above cases because the material and/or compaction specifications requirements in the Standard Specifications will not provide the desired results. Alternative engineering solutions should be utilized in these situations. Alternative engineering solutions include slope reconstruction, compacted fill, or use of stabilized material. Some alternatives to shoulder backing may require developing a nonstandard special provision.

672.2 Alternate Materials and Admixtures

(1) *Alternate Materials.* Alternate materials for shoulder backing include imported borrow and asphalt grindings.

(a) Imported Borrow: If native material does not meet the specifications for shoulder backing material, utilize imported borrow which meets the specifications for shoulder backing material.

(b) Asphalt Grindings: The Deputy Directive on Recycling Asphalt Concrete allows the use of asphalt grindings for shoulder backing; however, there are some limitations to where asphalt grindings can be used. For information on where asphalt grindings cannot be used consult the District Environmental unit. As stated in the Project Development Procedures Manual (PDPM), a Memorandum of Understanding (MOU) dated January 12, 1993 between the Department of Fish and Game (DFG) and Caltrans, allows Caltrans to use asphalt grindings for shoulder backing where these materials will not enter the water system.

(2) *Admixtures.* Admixtures may be used if recommended by the District Materials Engineer and their use is permitted in the environmental document and regulatory permits. District Environmental can assist in determining if and where admixtures can be used. Three types of admixtures (lime, cement, and seal coat with an asphaltic emulsion) are approved for use with shoulder backing.

(a) Lime and Cement Admixtures: Lime and cement are uniformly mixed into the shoulder backing material prior to application.

(b) Seal Coats: Seal coats with an asphaltic emulsion are applied in situ on top of shoulder backing material. When seal coats are specified, the appropriate seal coat special provisions should be included into the project special provisions. Seal coats are paid for separately from shoulder backing material.

672.3 Design

The limits, slopes, and other design details for shoulder backing need to be documented on the plans. The following design standards apply when designing shoulder backing details:

(1) Place shoulder backing from the edge of pavement (EP) to hinge point (HP). However, where the horizontal distance from EP to HP is greater than 3 feet, shoulder backing should be placed on a width of at least 2 feet from EP (See Figures 672.3A and 672.3B). The Design Engineer should consult with the District Materials Engineer for conditions where the distance from EP to HP is less than 2 feet and there are minimum hinge width requirements for dike, guardrail and barriers.

(2) Shoulder backing cross slope should be 10:1 or flatter where possible. Where there is insufficient width for a 10:1 slope, a steeper cross slope can be used but should not be steeper than 6:1 (See Figure 672.3A).

(3) The minimum hinge width (HW) from EP to new HP should be 2 feet (See Figure 672.3B). Where the existing HW is less than 2 feet, slope reconstruction (See Figure 672.3C) or some other strategy should be used instead of shoulder backing.

(4) Do not place shoulder backing on existing side slopes where shoulder backing cross slope will be steeper than 6:1 and/or the HW will be less than 2 feet (See Figures 672.3A & 672.3B).

(5) The maximum thickness for shoulder backing is 0.50 foot (See Figure 672.3B). Where the thickness will exceed 0.50 foot, use alternative strategies that have a combination of more stringent material and compaction requirements.

(6) Where the combined distance for HW and side slope will exceed 5 feet in order to comply with the slope requirement specified in this document, side slope reconstruction is recommended in lieu of shoulder backing (See Figure 672.3C).

(7) At the option of the District, shoulder backing can be placed up to a thickness of 0.50 foot to cap new construction or reconstructions (See Figures 672.3B & 672.3C).

(8) Place shoulder backing to match the pavement surface, even when the surface course layer is open graded friction course (OGFC). This reduces future maintenance needs to replace the shoulder backing as it subsides.

Figures 672.3A through 672.3E show some examples of what should and should not be done when using shoulder backing.

Figure 672.3A

Typical Application of Shoulder Backing

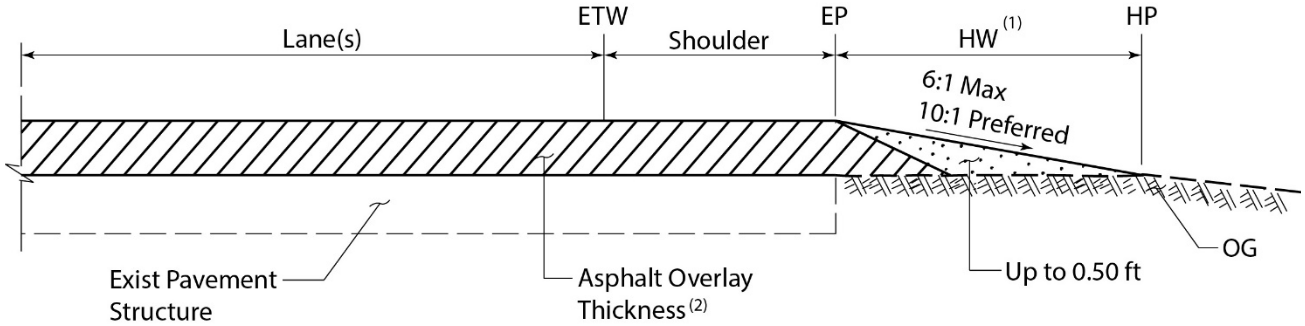
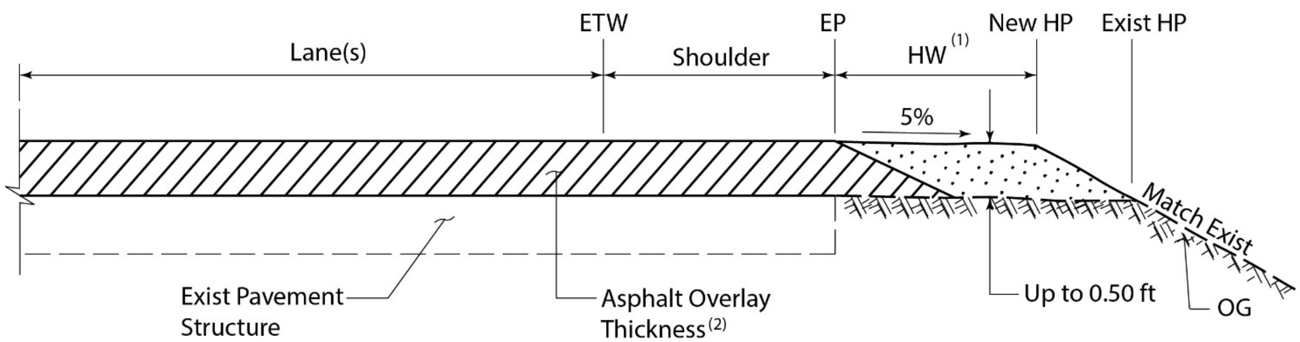


Figure 672.3B

Alternative Placement for Existing Slopes Steeper than 6:1



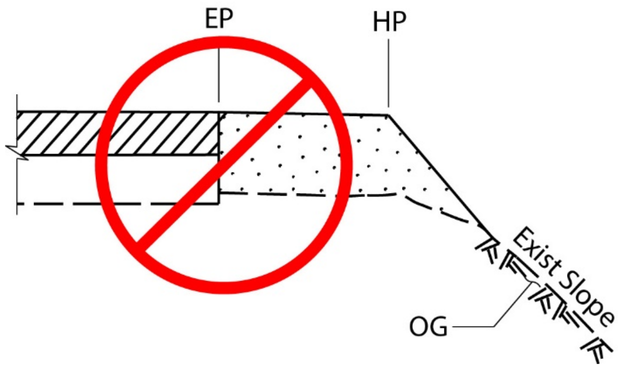
NOTES:

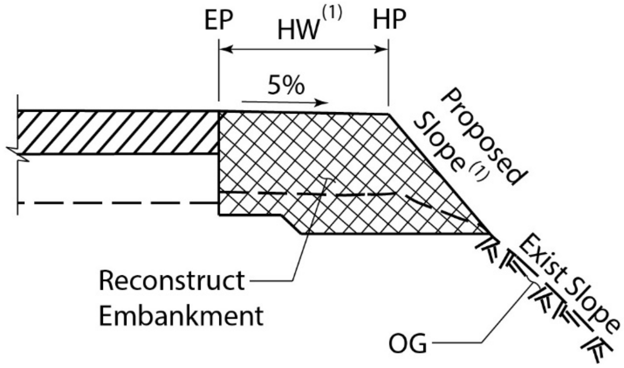
(1) Minimum Hinge Width (HW) is 2 feet. When HW is less than 3 feet, District Materials Engineer should be consulted regarding structural stability due to width reduction.

(2) Edge treatment shown are for asphalt overlay thickness of 0.45 foot or less. For asphalt thickness of more than 0.45 foot, see Standard Plans for edge treatment details.

Figure 672.3C

Placement of Shoulder Backing Thickness Greater Than 0.50 foot for Slope Repair





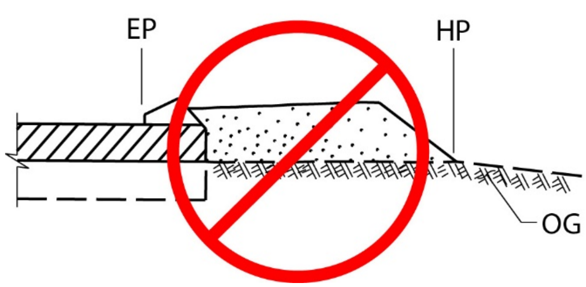
NOTES:

(1) See HDM Topic 304 for additional information on side slopes. See Standard Specifications for additional information on side slope construction.

See District Materials Engineer for material recommendations. (Roadway Geotechnical also needs to be consulted for slopes steeper than 2:1.).

Figure 672.3D

Placement of Shoulder Backing Behind Dikes



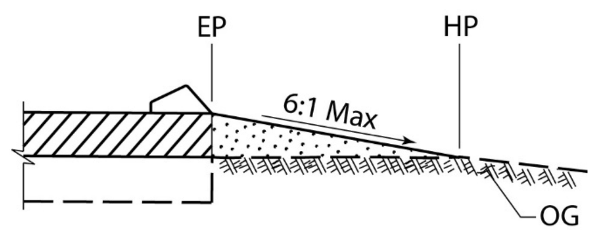
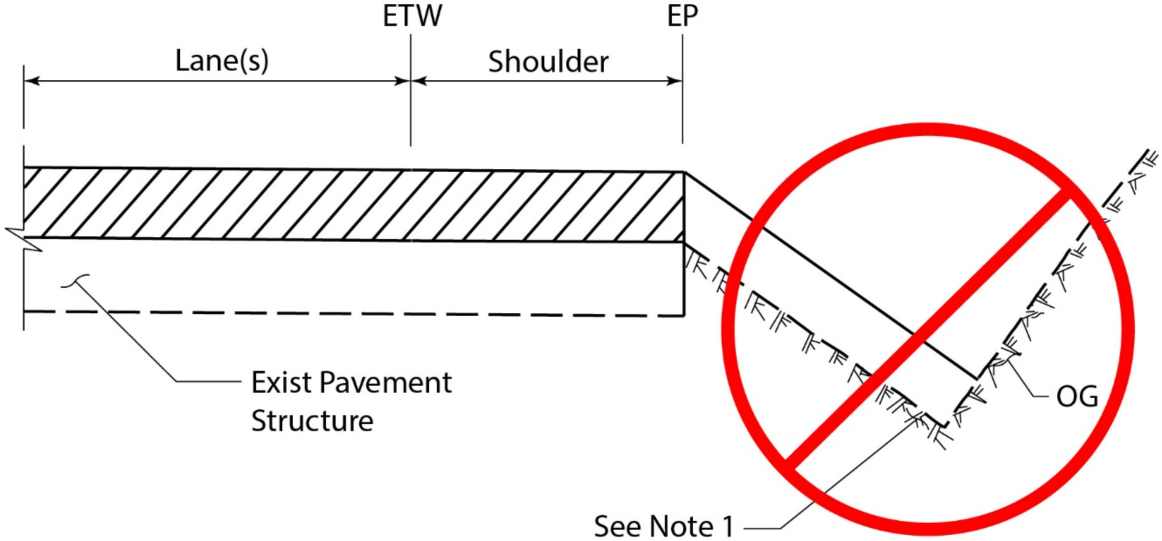


Figure 672.3E

Longitudinal Drainage (Roadside Ditches/Gutters)



NOTES:

(1) Consult with area Maintenance personnel and District Materials Engineer regarding erodability of ditch, alternative materials to shoulder backing, slope sloughing, and rockfall catchment in ditch.

Consult with District Hydraulics Engineer regarding acceptable change in ditch capacity.

Consult with District Stormwater Coordinator regarding water quality issues.

CHAPTER 700 MISCELLANEOUS STANDARDS

Topic 701 - Fences

Index 701.1 - Type, Intent and Purpose of Fences

(1) *Purpose of Fences.* Fences constructed by the Department serve the purposes of either establishing control of access, providing visual demarcation or re-establishing private property lines.

Where the purpose of the fence is access control, installation is intended to establish that access is restricted; such fencing is not intended to serve as a complete physical barrier. The adjacent private property owner will assume responsibility for the construction of any fencing or other facilities necessary to contain their personal property.

(2) *Type and Intent of Fences.* The type and intent of fences should be as described herein and in the Standard Plans and Standard Specifications.

Fence materials, including gates, installed anywhere within the State right of way are considered Departmental fences and are owned, controlled and maintained by Caltrans forces.

As a right of way consideration, Caltrans may construct fences and gates outside the State right of way. Fences and gates constructed outside the State right of way are considered private fences and are owned, controlled and maintained by the external property owner where Caltrans retains neither rights nor obligations for such fences once constructed.

(a) Fences for freeway and expressway access control are Departmental fences commonly placed immediately inside the State right of way to help enforce observance of the acquired access rights. See Index 701.2 for more detailed guidance.

(b) Median fences are Departmental fences constructed to help prevent indiscriminate crossings of the median by vehicles or pedestrians. These fences are a subset of freeway and expressway access control fences. See Index 701.2 for more detailed guidance.

(c) Private fences may be constructed adjacent to conventional highways if provided via right of way agreement. Placement is typically parallel to the State right of way and outside Caltrans property. See Index 701.3 for more detailed guidance.

Private fences may also be allowed within Caltrans right of way to restrict access to a private facility crossing or as an aesthetic enhancement of Departmental fence. Neither of these situations is common and should be avoided if possible. See Indexes 701.2(3)(e) and 701.3.

(d) Temporary fences are commonly used during project construction to temporarily control access and/or create a visual screen. Temporary fences are also commonly used during reconstruction of either Departmental or private fences. See Index 701.4 for more detailed guidance.

(e) Environmentally Sensitive Area (ESA) fence is a specialty type of temporary Departmental fence, placed within the limits of a construction project and used to identify the location of sensitive biologic resources while establishing a visible boundary. Orange fabric is used to ensure contractor personnel awareness of the ESA location. See Index 701.5 for more detailed guidance.

(f) Species protection fences are Departmental fences placed within Caltrans right of way and used to prohibit movement of specific threatened or endangered species onto the highway. These fences are unique in composition to the species being addressed. Species protection fences may be placed for either permanent or temporary applications. See Indexes 701.2(3)(b) and 701.5 for more detailed guidance.

(g) Enclosure fences are Departmental fences of various types used to secure the perimeter around equipment storage areas from theft or vandalism, provide a perimeter around maintenance stations or other facilities, or otherwise enclose areas intended for Caltrans use. See Index 701.5 for more detailed guidance.

(3) *Approval.* The District Director has the authority and responsibility for approval of fence type and location within the standards stated herein.

701.2 Freeway and Expressway Access Control Fence

(1) *Placement.* **Departmental fences shall be provided on freeways and expressways to control access, except as otherwise provided under paragraph (3)(e) below.** Freeway fencing or equivalent access control should extend to the limit of the legal access control on local streets at ramp termini.

(2) *Standard Fence Types.* The standard types of freeway fence are:

(a) Chain Link Fencing--Type CL-6 fence or equivalent access control should be used along the right of way and in the outer separation in urban or developed areas.

(b) Other Fencing--In rural areas, fences on freeways normally should be either Barbed Wire, (Type BW), or Wire Mesh, (Type WM), on either wood or metal posts. Wood posts may be more aesthetic than metal posts, depending on the surrounding terrain.

(c) Median Fencing--Type CL-4 fence, with the distance from the ground to the bottom tension wire increased to 6 inches, should be used where median fencing is required.

(3) *Exceptions to Standard Fence Types.*

(a) If walls or fences equal to or better than the standard fence in durability, maintenance requirements, and dimensions exist along the right of way line, the standard fence may be omitted or removed. To avoid a gap in the access control, standard fences should be securely joined to the existing fence or wall at its terminals, if the access control line extends beyond these points.

(b) Fences of special design may be installed where needed for wild animal control.

(c) In special cases, where improvements are scattered, the area is aesthetically sensitive, and a lower fence would be in keeping with the height of adjacent property fence, a Type CL-4 fence may be substituted for Type CL-6 along the right of way in locations where Type CL-6 would otherwise be used.

(d) Fencing may be omitted in remote areas where access control appears unnecessary.

(e) In special cases, nonstandard fencing may be considered at freeway ramp terminals on local streets when the adjacent property either is, or is proposed to be, developed in such a way that the owner feels that standard fencing is aesthetically objectionable. If it is concluded that the objection is valid, a more compatible facility may be substituted, subject to the following controls:

- Preference should be given to retaining the standard fence along the ramp to the end of the curb return or beginning of the taper on the local road. Where this is not reasonable, there may be substituted a fence or wall of equal or better durability and utility that is at least 4 feet high relative to the grade of freeway right of way line. Walls, ornamental iron fences with closely spaced members, or chain link fences are examples of acceptable possibilities.

- Along the local road, beyond the end of the curb return or the beginning of the taper, a facility of somewhat lower standards may be employed, if considered appropriate. The minimum allowable height is 2.5 feet above the grade at the edge of the right of way. In addition to the fence types suitable for use along the ramp, split rail fences, wooden picket fences, and permanent planter boxes are examples of possibilities. The intent is to delineate the access control line and discourage access violations in an effective manner.

- Generally, all costs for the removal of the existing freeway fence and the installation and future maintenance of a nonstandard fence are to be the property owner's responsibility under the terms of the encroachment permit authorizing the substitution. On new construction, the property owner is to assume similar costs and responsibilities subject to a credit for the value of a standard fence.

(4) *Location of Fences.* Normally, fences on freeways should be placed adjacent to, but on the freeway side of the right of way line.

Fences in the outer separation normally should be placed as shown in Figure 307.4 so that the area outside of the fence may be relinquished to the local agency.

When viewed at a flat angle, chain link fencing restricts sight distance. This fact should be considered in the location of such fencing at intersections. To eliminate hand maintenance, right-angle jogs should be avoided.

(5) *Locked Gates.* Locked gates may be provided in access control fences in special situations. A proposal for a locked gate must address a necessity. Although openings controlled by locked gates do not constitute access openings in the usual sense of access control, they must be shown on the plans. When locked gates are proposed there must be a specific reason for each gate. All gates must be kept locked and secured. Locked gates fall into two categories:

(a) Locked gates to be used exclusively for access by highway maintenance forces do not require FHWA approval and may be approved by the District Director. The integrity and security of this access must always be assured. Maintenance forces must also keep gates locked when not being used for the access of persons or equipment. When locked gates are to be used exclusively by highway maintenance forces, one or more of the following criteria apply:

- A circuitous route would be eliminated.
- The gate access would minimize the exposure of maintenance workers to highway traffic.
- Parking is available outside the gate.
- The gate would allow slow moving equipment to be kept off the highway.
- The site is not accessible to maintenance personal or equipment from the freeway.

(b) Proposals for locked gates to be used by utility companies must be submitted to the District Director for approval. The gate submittal must present all pertinent facts and alternate solutions.

Locked gates to be used by other public agencies or by non-utility entities require FHWA approval if the gate is on an Interstate route.

When proposals for locked gates requiring FHWA approval are included in the plans for new construction, including landscaping projects, FHWA approval of such gates will be included in FHWA approval of the project PS&E. Subsequent installations requiring FHWA approval must be submitted separately to FHWA by the Division of Design after approval by the Chief, Division of Design.

701.3 Private Fences

(1) *Placement.* Caltrans will construct or pay the cost of fences on private property only as a right of way consideration to mitigate damages. Caltrans' construction of such fences should be limited to:

(a) The reconstruction or replacement of existing fences.

(b) The construction of fences across property that had been previously enclosed by fences.

These criteria apply to all private as well as public lands.

(2) *Private Fences Inside the State Right of Way.* Private fences may be constructed within the State right of way via Encroachment Permit to restrict access to facilities (e.g., canals)

crossing under or through Department-owned property. A Maintenance Agreement must be executed to provide for future maintenance of the fence and allow access to the private utility.

701.4 Temporary Fences

(1) *Placement.* Temporary fences are located where necessary in accordance with construction contractor activities and where the right of way rights have been acquired.

(2) *Types of Fences.* Temporary fence design should conform to the needs of the situation and the length of time to be used. In most access control or demarcation applications the fence fabric will conform to permanent fence standards, while lesser requirements may apply to posts and post footings to more readily accommodate removal when no longer needed.

Temporary fence used during reconstruction of private fences must be of a type adequate to meet the permanent private fence purposes.

701.5 Other Fences

(1) *ESA and Species Protection Fences.* District Environmental Unit staff must specify the required placement limits and locations for ESA and species protection fences.

ESA fence material requirements are described in Section 14 of the Standard Specifications.

Species protection fences will be uniquely designed to meet the needs of the target species. District Environmental staff will provide information on the necessary design parameters. In many instances, species protection fence will be able to be directly attached to existing freeway or expressway access control fence and thus preclude the need for separate posts. Where species protection fence is to be constructed along conventional highways, it must be constructed inside the State right of way and should not be attached to any private fence that may exist.

(2) *Enclosure Fences.* Because these fences are commonly intended to provide security for Caltrans facilities, the facility type and location will often dictate the fence design to be used. Standard chain link (CL-6) fence is most common, but additions (barbed wire extension arms) or alternative designs may be considered. When slats are included as an element of the design, wind forces must be considered and there will be a resulting increase in the size and depth of embedment of fence posts as well as an increase in the size of the concrete footing. Table 701.5 provides recommended post size and embedment along with footing size for CL-6 slatted fence under an assumption of relatively weak soil resistance (indicated as "unconstrained") as well as for situations where the fence is installed through paved areas (common at maintenance stations, indicated as "constrained"), and a design wind velocity of 105 mph. For differing fence heights, wind velocities, or soil conditions, special analysis may be warranted. Contact the Office of Highway Drainage Design in Headquarters for assistance.

Table 701.5

Slatted CL-6 Post & Footing Dimensions

Condition	Post NPS (Standard Cut)	Footing	
		Dia.	Depth
Unconstrained	4"	18"	3'-6"
Constrained	4"	18"	5'-6"

Typically District Maintenance or Traffic Operations will specify any unique design requirements for enclosure fences as they will assume responsibility after construction.

Topic 702 - Miscellaneous Traffic Items

702.1 References

(1) *Guardrail and Crash Cushions.* See Chapter 7 of the Traffic Manual.

(2) *Markers.* See Part 3 of the California Manual on Uniform Traffic Control Devices (California MUTCD).

(3) *Truck Escape Ramps.* See Traffic Bulletin No. 24, (1986) and the NCHRP Report 178.

(4) *Mailboxes.* See the AASHTO Roadside Design Guide, 3rd Edition, Chapter 11, "Erecting Mailboxes on Streets and Highways".

Topic 703 - Special Structures and Installation

703.1 Truck Weighing Facilities

The Division of Traffic Operations coordinates the design and construction of truck weighing facilities with the California Highway Patrol in Sacramento. Typical plans showing geometric details of these facilities are available from the Headquarters Division of Traffic Operations. Districts should refer truck weighing facility maintenance issues to their District maintenance units.

See Index 107.1 for additional details on roadway connections for truck weighing facilities.

703.2 Rockfall Restraining Nets

Rockfall Restraining Nets are protective devices designed to control large rockfall events and prevent rock from reaching the traveled way. The systems consist of rectangular panels of woven wire rope vertically supported by steel posts and designed with frictional brake elements capable of absorbing and dissipating high energies. For additional information on the characteristics and applications for rockfall restraining nets, designers should contact the Division of Engineering Services - Geotechnical Services (DES-GS).

Topic 704 - Contrast Treatment

704.1 Policy

In general, delineation should be composed of the standard patterns discussed in Part 3 of the California MUTCD.

Markings include lines and markings applied to the pavement, raised pavement markers, delineators, object markers, and special pavement treatments.

Contrast treatment is designed primarily to provide a black color contrast with an adjacent white surface. Normally, contrast treatment should be used only in special cases such as the following:

(a) To provide continuity of surface texture for the guidance of drivers through construction areas.

(b) To provide added emphasis on an existing facility where driver behavior has demonstrated that standard signs and markings have proven inadequate.

When contrast treatment is applied, a slurry seal should be used.

See Part 3 of the California MUTCD for additional information on contrast treatment.

Topic 705 - Materials and Color Selection

705.1 Special Treatments and Materials

Special materials or treatments, such as painted concrete, or vinyl-clad fences, are sometimes proposed for aesthetic reasons, or to comply with special requirements.

The following guidelines are to be used for the selection of these items:

(a) Concrete should not be painted unless exceptional circumstances exist, due to the continuing and expensive maintenance required. Concrete subject to unintentional staining should be textured during construction to minimize the visibility of stains, if other methods of controlling stain-producing runoff or dripping cannot be accomplished.

(b) Vinyl-clad fences are sometimes specified for aesthetic reasons. The cost of this material is higher than that of galvanized steel. Special consideration should be given to the life-cycle cost and maintainability of vinyl-clad fencing prior to selection for use. The use of black or green vinyl-clad mesh for access control fencing, safety fencing at the top of retaining walls, and pedestrian overcrossing fencing is acceptable.

705.2 Colors for Steel Structures

Colors for steel bridges and steel sign structures may be green, gray, or neutral tones of brown, tan, or light blue.

Criteria for selection of colors are:

(a) General continuity along any given route.

(b) Coordination of color schemes with adjacent Districts for interdistrict routes.

(c) Requests from local agencies for improvement of aesthetics in their community.

Color selection for steel bridges should be mutually satisfactory to the Division of Engineering Services and the District. The Division of Engineering Services (DES) will initiate the color selection process by submitting the proposed color to the District Landscape Architect for review. The color for steel sign structures will be selected by the District Landscape Architect.

Topic 706 - Roadside Treatment

706.1 Roadside Management

A key concept in roadside management is that roadway and roadside design should consider the full life-cycle cost of transportation improvements including the long-term cost of maintenance. The design alternative with the lowest initial construction cost may not be the best solution if this approach will include high recurring maintenance costs. Designers should strive to select design approaches that do not require extensive recurring long-term activities.

A second key roadside management concept is that roadway and roadside design should contribute to the safety of Department maintenance workers by incorporating techniques that eliminate or reduce worker exposure to traffic. More specifically, these management concepts include the following techniques:

- Eliminate the need for recurrent maintenance activities such as vegetation control, herbicide application, pruning, mowing and graffiti removal;
- Facilitate the automation of recurrent maintenance activities such as herbicide application, mowing and litter collection;
- Locate facilities that require recurrent maintenance activity outside the clear recovery zone, or within protected areas;
- Provide safe maintenance worker access to facilities that require recurrent maintenance activity.

To implement this second roadside management concept, the following conditions must be considered in roadway and roadside design projects:

- Metal beam guardrail, including standard railing, terminal system end treatments, guard railing at structure approach and departures, and at fixed objects should include vegetation control. For more detailed information regarding placement of vegetation control consult with both the District Landscape Architect and District Maintenance. See the Standard Plans for minor concrete vegetation control.
- Thrie beam barrier, including single thrie beam barrier, double thrie beam barrier, at structure approach and at fixed objects should include vegetation control. For more detailed information regarding placement of vegetation control consult with both the District Landscape Architect and District Maintenance. See the Standard Plans for minor concrete vegetation control.
- Unpaved narrow strips often result from the construction of noise barriers or concrete barriers beyond the paved shoulder edge. Unpaved strips 15 feet or less in width, parallel and immediately adjacent to the roadway, should be paved to the barrier or wall. Paving these areas eliminates the need for manual vegetation control, and allows automated equipment to remove litter and debris. Pavement requirements are consistent with the guidance contained in this manual. Contrasting surface treatment such as markings, delineation, or color may also be provided so drivers can distinguish these areas from those intended for vehicular use.
- Unpaved areas greater than 15 feet in width may include vegetation control techniques such as weed control mats, patterned asphalt or stamped concrete paving, or the planting of low maintenance vegetation such as native grasses. Consult the District Landscape Architect and District Maintenance to select and appropriate vegetation control technique.
- Plants, which at maturity may encroach upon required site distances, should be removed. Consult the District Landscape Architect to identify potential encroaching plant material.

- Noise barriers should be designed with a textured aesthetic treatment or planted with vines to reduce maintenance required to control graffiti. Index 902.3 of this manual and the Project Development Procedures Manual contain information of the planting on noise barriers.
- Unpaved area beyond the gore pavement should be paved as per Index 504.2(2).
- Roadside facilities that require recurring maintenance, such as irrigation controllers, electrical controllers, backflow preventers, and valve boxes, should not be placed on the outside of horizontal curves, near gore areas, near auxiliary lanes, or near ramp termini. The designer should strive to place these facilities outside the clear recovery zone, or within a protected area if placement outside the clear recovery zone is not feasible.
- When placing roadside facilities that require recurring maintenance, the designer should strive to include improvements that facilitate safe maintenance access such as maintenance vehicle pullouts, maintenance access paths, walk gates and vehicle gates. It is preferred that access be provided from outside the right-of-way for all facilities that require maintenance access.
- When placing noise barriers in areas with a narrow right of way, the designer should consider locating a concrete safety shape barrier 3 feet from the face of the noise barrier to provide protected maintenance access to planting and irrigation facilities.

Formal safety reviews for roadside management issues should be accomplished as discussed in Index 110.8. Consult the District Landscape Architect and District Maintenance unit early during design development to identify and address potential roadside management issues, such as avoiding the redundant placement of roadside facilities, or allow for the consolidation of roadside facilities.

706.2 Vegetation Control

Weed control fabric or soil sterilant chemicals may be placed under pavement to prevent weed growth through medians, traffic islands, and other paved areas.

The Division of Maintenance is responsible for the selection of herbicides. Approval is required for any changes from the currently approved Standard Specifications and Standard Special Provisions for pesticides and herbicides.

Since soil sterilants may be transported by water, they should not be used where they may affect environmentally sensitive areas, habitat, native vegetation, landscape plantings, agricultural crops, adjacent residential, commercial or recreation areas, streams, or water bodies.

Before specifying soil sterilants, the District Landscape Architect should be consulted to determine the possibility of future planting.

706.3 Topsoil

In areas of new construction, quality existing topsoil should be stockpiled and spread during the final stages of construction. The native brush should be crushed or chipped and mixed with the stockpiled soil to maximize natural or organic matter in the soil. Since topsoil contains beneficial microorganisms and seed, it is best to stockpile it in shallow windrows and planted with temporary erosion control so that oxygen can penetrate the soil.

706.4 Irrigation Crossovers for Highway Construction Projects

Irrigation crossovers normally consist of a conduit with a waterline crossover and sprinkler control conduit with pull wire. Irrigation crossovers should be provided under new roadways and ramps when future highway planting is anticipated. The District Landscape Architect should be consulted to determine the need for such crossovers as well as size and location. Attention should also be given to extending existing conduits when widening or modifying roadways and ramps.

The following factors should be considered in sizing and locating crossovers:

(a) A standard irrigation crossover consists of a minimum size of 8-inch diameter nominal (DN) conduit, with a 3-inch DN water supply line and a 2-inch DN sprinkler control conduit with pull wire. Sizes of irrigation crossovers and water supply lines are usually larger when nonpotable water is to be used.

(b) Irrigation crossovers are typically spaced 1,000 feet apart on freeways where future highway planting is anticipated. Undercrossings may be considered alternative crossing opportunities.

(c) Drainage facilities should not be used for waterline crossings.

Standard details and special provisions for the irrigation crossover should be furnished by the District Landscape Architect to the Project Engineer for highway construction projects.

706.5 Water Supply Line (Bridge) and Sprinkler Control Conduit for Bridge

Water supply line and sprinkler control conduit with pull wire should be provided in new bridge structures.

The District Landscape Architect should be consulted to determine the need for such water supply lines and sprinkler control conduits such as size and location.

Attention should also be given to modifying, changing existing, or installing new water supply lines and sprinkler control conduits when widening or modifying bridge structures.

The following factors should be considered in sizing and locating water supply lines and sprinkler control conduits:

(a) Generally, locate on the side of the bridge, nearest the water source.

(b) Consider the maximum water demand and number of irrigation controller stations anticipated to be used. The water supply line should be a minimum 3-inch DN and the conduit for the sprinkler control conduit should be a minimum 2-inch DN and contain a pull wire.

(c) Ductile iron pipe is required for the water supply line for pipes 4-inch DN or larger because of its superior strength and flexible joints.

706.6 Water Supply for Future Roadside Rest Areas, Vista Points, or Planting

Provision for a permanent water supply should be included in the major construction project. In the preparation of a major highway construction project, consideration should be given to using the water source needed for construction as part of a future permanent water supply system. If this appears to be a feasible solution, consider such factors as:

(a) Probability of a future planting, vista point, or roadside rest project.

(b) Economy.

(c) Possible reduction in the flexibility of the highway contractor's operation.

The District Landscape Architect should be consulted.

Topic 707 - Slope Treatment Under Structures

707.1 Policy

Structure end slope should be treated to:

(a) Protect slopes from erosion.

(b) Improve aesthetics.

(c) Reduce long term maintenance costs.

Caltrans maintenance, landscape architecture, materials, design, and other affected units will furnish input to determine slope treatment needed at each site. Local agency input should be obtained for urban undercrossings.

All types of slope treatments require adequate drainage facilities for water from the upper roadway. Inadequate drainage is a major source of slope erosion.

707.2 Guidelines for Slope Treatment

(a) Full slope paving shall be installed where it is anticipated that erosion by pedestrians, wind, storm water, or other causes will occur. High landscape maintenance costs caused by inadequate moisture, sunlight, instability to establish vegetation etc., may also justify the use of full slope paving in lieu of planting. The District Landscape Architect will provide aesthetic input and waterline crossover conduit as well as locations for slope paving.

(b) Landscaped structure end slopes may be justified when adjacent slopes are landscaped and when landscaping is compatible with adjacent development. Conditions must exist where plants would have a strong likelihood of survival.

(c) Bare slopes have minimum initial costs and higher maintenance costs which vary with the site. Bare structure end slopes may be justified at rural sites and other areas where anticipated maintenance activity will be low and there is little likelihood for erosion. Appropriate drainage design is critical when slopes are left bare.

(d) Adequate drainage facilities must be provided to prevent saturation of abutment foundation materials and damage to slope treatment.

(e) Additional protection may be required at stream crossings to provide for flow velocity.

707.3 Procedure

Based on consultation with the District Landscape Architect and Structures Bridge Architect and in consideration of economic and aesthetic factors, the District will determine, and set forth with the bridge site plan submittal, the type of slope treatment indicating whether:

(a) The Division of Engineering Services is to design the slope treatment with the bridge and include the cost in the Structure items; or

(b) The District will design the slope treatment and include the details with the road plans.

CHAPTERS 800 - 890
HIGHWAY DRAINAGE DESIGN

CHAPTER 800 - GENERAL ASPECTS

Topic 801 - General

Index 801.1 - Introduction

This section is not a textbook, and is not a substitute for fundamental engineering knowledge or experience.

The fields of hydrology and the hydraulics of highway drainage are rapidly evolving and it is the responsibility of the engineer to keep abreast of current design practices. As new practices or procedures are adopted by the Department, this section will be updated.

Instructions for the design of highway drainage features provided are for information and guidance of Department employees. Drainage policies, procedures and standards given are subject to amendment as conditions warrant and are neither intended as, nor do they establish, legal standards. Special situations may call for variations from these requirements, subject to approval of the Division of Design or approval by others as may be specifically referenced.

801.2 Drainage Design Philosophy

Highway drainage design is much more than the mere application of the technical principles of hydrology and hydraulics. Good drainage design is a matter of properly balancing technical principles and data with the environment giving due consideration to other factors such as safety and economics. Such design can only be accomplished through the liberal use of sound engineering judgment. Drainage features to remove runoff from the roadway and to convey surface and stream waters originating upstream of the highway to the downstream side should be designed to accomplish these functions without causing objectionable backwater, excessive velocities, erosion or unduly affecting traffic safety. A goal in highway drainage design should be to perpetuate natural drainage, insofar as practical.

801.3 Drainage Standards

Drainage design criteria should be selected that are commensurate with the relative importance of the highway, associated risks, and possible damage to adjacent property. The objective of drainage design should be to provide optimum facilities considering function versus cost rather than to just meet minimum standards.

Engineers and other professional disciplines using this guide must recognize that hydrologic analysis, as practiced by the highway engineer, has not advanced to a level of precise mathematical expression. All hydrologic analysis methods, whether deterministic or statistical, are based on the information available. A common challenge faced by the highway design engineer is that there may be insufficient flow data or no data at all at the site for which a stream crossing is to be designed. By applying analytical principles and methods it is possible to obtain peak discharge estimates which are functionally acceptable for the design of highway drainage structures and other features.

The design of highway drainage structures and other features must consider the probability of flooding and provide protection which is commensurate with the importance of the highway, the potential for property damage, and traffic safety. Traditionally, the level of assurance for such protection has been specified in terms of the peak rate of flow during passage of a flood or storm of the severity associated with the frequency of occurrence, i.e. a 10-year storm, the 50-year flood, etc. State-of-the-art methods and procedures associated with the necessary hydrologic analysis required to determine the severity and probability of occurrence of possible rare storms and flood events are inherently ambiguous. Therefore, the suggested drainage design criteria relating to frequency of occurrence references in this manual are provided for guidance only and are not intended to establish either legal or design standards which must be strictly adhered to. Rather, they are intended as a starting point of reference for designing the most cost effective drainage structures and facilities considering the importance of the highway, safety, legal obligations, ease of maintenance, and aesthetics.

801.4 Objectives of Drainage Design

Drainage design seeks to prevent the retention of water by a highway and provide for removal of water from the roadway through a detailed analysis considering all pertinent factors.

Specific steps to be taken generally include:

(a) Estimating the amount and frequency of storm runoff.

(b) Determining the natural points of concentration and discharge, the limiting elevations of entrance head, and other hydraulic controls.

(c) Estimating the amount and composition of bedload and its abrasive and bulking effects.

(d) Determining the necessity for protection from floating trash and from debris moving under water.

(e) Determining the requirements for energy dissipation and bank protections.

(f) Determining the necessity of providing for the passage of fish and recognizing other ecological conditions and constraints. Water quality and pollution control are discussed under Index 110.2. Aspects of wetlands protection are covered under Index 110.4.

(g) Analyzing the deleterious effects of corrosive soils and waters on structures.

(h) Comparing and coordinating proposed design with existing drainage structures and systems handling the same flows.

(i) Coordinating, with local agencies, proposed designs for facilities on roads to be relinquished.

(j) Providing access for maintenance operations.

(k) Providing for removal of detrimental amounts of water on traveled ways (see Topics 831 and 833).

(l) Providing for removal of detrimental amounts of subsurface water.

(m) Designing the most efficient drainage facilities consistent with the factors listed above, economic considerations, the importance of the transportation facility, ease and economy of maintenance, engineering judgment, and aesthetics.

(n) Checking the structural adequacy of designs by referral to Structures Design or by use of data furnished by Structures Design.

(o) Preventing water from crossing slopes in concentrated flows.

801.5 Economics of Design

An economic analysis of alternate drainage designs, where a choice is available, should always be made. Non-engineering constraints may severely limit the design alternatives available to the drainage design engineer for a specific project or location. Generally, however, the design engineer has a wide range of materials and products to choose from in selecting the most economical design from available alternatives for highway drainage structures and other features.

The following factors should be considered in the selection of alternative designs and economic comparisons:

(a) Initial cost of construction and right of way.

(b) Evaluation of flood related risks to the highway and to adjacent properties including potential liabilities for damage.

(c) Cost of detours and traffic handling.

(d) Service life of the highway and of the drainage structure.

(e) Cost of providing traffic safety features.

(f) Aesthetics.

(g) Costs to traveling public for delays or extra travel distance due to road closures.

(h) Initial cost versus long term maintenance costs for cleanout, repair, traffic control and other pertinent maintenance charges that may be incurred during the life of the facility.

(i) Safety of required maintenance activities, ability to provide maintenance mechanically and to reduce worker exposure.

(j) Inlet and outlet treatment.

(k) Potential for causing erosion and effective water pollution control.

801.6 Use of Drainage References

No attempt has been made herein to detail basic hydrologic and hydraulic engineering techniques.

Various sources of information, including FHWA Hydraulic Engineering Circulars (HEC's); Title 23, Code of Federal Regulations (CFR), Part 650, Subpart A; AASHTO Guidelines; Federal-Aid Policy Guide and numerous hydrology and hydraulics reports and texts have been used to compile this highway drainage guide. Frequent references are made to these publications. Where there is a conflict in information or procedure, engineers must look at all pertinent parameters and use their best judgment, to determine which approach is the most consistent with the objectives of Caltrans drainage design principles and which most closely relates to the specific design problem or project.

Topic 802 - Drainage Design Responsibilities

802.1 Functional Organization

(1) Division of Design. The Office of State Highway Drainage Design in Division of Design performs the following functions under the direction of the Headquarters Hydraulics Engineer:

(a) Provide design information, guidance and standards to the Districts for the design of surface and subsurface drainage.

(b) Keep informed on the latest data from research, experimental installations, other public agencies, and industry that might lead to improvement in drainage design practices.

(c) Promote statewide uniformity of design procedures, and the exchange of information between Districts.

(d) Coordinate drainage design practices with other Caltrans Offices.

(e) Review special drainage problems and unusual drainage designs on the basis of statewide experience.

(f) Act in an advisory capacity to the Districts when requested.

(2) Division of Engineering Services (DES). The DES is responsible for:

(a) The hydraulic design of bridges, bridge deck drains, and special culverts.

(b) The structural adequacy of all drainage facilities.

(c) The adequacy of pumping plant characteristics and temporary storage. Refer to Topic 839 for further discussion on pumping stations.

(d) Compliance with Federal-Aid Policy Guide, Transmittal 1, G 6012.1 and submittal of preliminary hydraulic data as outlined under Topic 805.

(e) Geotechnical (soil mechanics and foundation engineering) considerations.

(3) Legal Division. The Legal Division provides legal advice and guidance to other Caltrans Offices concerning the responsibilities of the Department and owners of property along State highways with regard to surface water drainage.

(4) Districts. The District Director is responsible for:

(a) The hydrology for all drainage features except bridges.

(b) The hydraulic adequacy of all drainage features, except bridges and any special culverts and appurtenances designed by the Division of Engineering Services.

(c) Consulting with the Division of Engineering Services when it is proposed that an existing bridge be replaced with a culvert.

(d) Bank and shore protection designs, including erosion protection measures at ends of bridges and other structures designed by the Division of Engineering Services.

(e) Assigning one or more engineers in responsible charge of hydrologic study activities and the hydraulic design of drainage features.

(f) Compliance with Federal-Aid Policy Guide, Transmittal 1, G 6012.1 for storm drain systems.

(g) Providing additional staff as necessary with the training and background required to perform the following:

- Accomplish the objectives of drainage design as outlined under Index 801.4
- Prepare drainage plans or review plans prepared by others.
- Study drainage problems involving cooperative agreements and make recommendations to the decision makers.
- Accumulate and analyze hydrologic and hydraulic data reflecting the local conditions throughout the District for use in design.
- Review drainage changes proposed during construction.
- Make investigations and recommendations on drainage problems arising from the maintenance of existing State highways.
- Coordinate drainage design activities with other District Offices and Branches.
- Coordinate drainage designs with flood control districts and other agencies concerned with drainage by representing the District at meetings and maintaining an active liaison with these agencies at all times.
- Furnish data as required on special problems, bridges, large culverts, culverts under high fills and pumping plants that are to be designed by the Division of Engineering Services.
- Make field inspections of proposed culvert sites, existing drainage structures during storms, and storm damage locations.
- Document condition and file data that might forestall or defend future lawsuits.
- Review permits for drainage facilities to be constructed by other agencies or private parties within the highway right of way.
- Investigate and prepare responses to complaints relative to drainage conditions on or adjacent to the right of way.

Assignment of the duties described above will vary between districts. Due to the increasing complexity of hydraulic and hydrologic issues it is imperative that the more complex analyses be performed by experienced hydraulic designers. To provide guidance on those issues where district hydraulic units should become involved, the following list is provided.

- Storm drain design and calculations.
- Drainage basins exceeding 320 acres.
- Hydrograph development or routing.
- Open channel modification or realignment.
- Retention or detention basins.
- Backwater analysis.
- High potential for flood damage litigation.
- Scour analysis or sediment transport (typically forwarded to DOS).
- Culvert designs greater than 36 inches in diameter.
- Encroachments on FEMA designated floodplains.
- Modifications to inlet or outlet capacities on existing culverts or drainage inlets (e.g., placement of safety end grates, conversion of side opening inlets to grated inlets, etc.).

- Unique hydraulic design features (e.g., energy dissipator design, pumping stations, siphons, etc.).

This list is not all inclusive, and many additional functions are likely to be performed by hydraulic units. Although various constraints may preclude the hydraulic unit from actively performing the design or analysis of these items, a thorough review by that unit should be performed, at a minimum.

(5) *Materials Engineering and Testing Services.* METS provides advice and guidance to other Caltrans Offices and Branches concerning service life, physical properties, and structural adequacy of materials used in drainage design.

802.2 Culvert Committee

The Caltrans Culvert Committee is composed of nine members representing the Offices of State Highway Drainage Design, Structure Design, Office Engineer, and Materials Engineering and Testing Services, along with the Division of Construction and the Division of Maintenance. The Committee is chaired by the Headquarters Hydraulics Engineer in the Office of Highway Drainage Design. The Committee performs the following functions:

(a) Investigates new materials and new installation methods that may improve the economic service life of culverts and other drainage facilities.

(b) Coordinates drainage design practice with other headquarters departments.

(c) Follows current research and takes steps to implement successful findings.

(d) Acts as an advisory group to Districts and other Caltrans Offices when requested.

(e) Serves as Caltrans liaison with manufacturers, suppliers, contractors and industry associations.

The authority of the Committee is advisory only, and recommendations of the Committee are submitted to the Chief, Division of Design for approval and implementation through design guidelines and standards.

Requests for consideration of new materials, methods, or procedures should be directed to the Committee Chairman.

802.3 Bank and Shore Protection Committee

The Caltrans Bank and Shore Protection Committee is composed of representatives from DES Structures Maintenance and Investigation, Office of State Highway Drainage Design, METS, Division of Construction, and Division of Maintenance. It is chaired by the Office of Highway Drainage Design representative.

The Committee performs the following functions:

(a) Acts as a service and an advisory group available to Districts and Caltrans Offices and Branches upon written request for special investigations or study. Requests for special investigation of rock slope protection, channel or bridge protection, major channel changes, etc. should be directed to the Committee Chair.

(b) Provides conceptual input and acts as approval authority for supplements or modifications to bank and shore protection practice publications as warranted.

(c) Investigates and provides input toward the development of detailed design criteria for the various types of bank and shore protection.

(d) Observes performances of existing and/or experimental installations during or following severe exposures. The Districts or Caltrans Offices or Branches are requested to inform the Chair, Bank and Shore Protection Committee, or any available members of the Committee, of damage to installations by flood or high seas.

(e) Upon submission by the Department's New Products Coordinator, the Committee evaluates new products and processes related to bank and shore protection for possible approval.

Topic 803 - Drainage Design Policies

803.1 Basic Policy

In drainage design, the basic consideration is to protect the department's facilities against damage from storm and subsurface waters, taking into account the effect of the proposed improvement on travelers and property. Unless the State would benefit thereby, or the cost is borne by others, no improvement in the drainage of areas outside the right of way is to be considered on Caltrans projects.

803.2 Cooperative Agreements

The extent of the department's financial participation in cooperative drainage improvement projects must be commensurate with the benefits to the Department and the traveling public.

(1) *Local Agencies.* Caltrans may participate with Local Agencies, Flood Control Districts or Drainage Assessment Districts on drainage improvement projects. Such projects must be covered by a formal agreement prepared and processed in accordance with instructions in the Caltrans Cooperative Agreement Manual.

(2) *Federal and State Flood Control Projects.* The cost of upgrading or modifying existing State highway facilities to accommodate Federal and/or State funded flood control projects is normally the responsibility of the agency funding the project. As necessary, Caltrans may enter into agreements containing provisions that the cost of betterments to existing highways, including drainage features, will be paid for by the Department. The Cooperative Agreement Manual contains procedures for preparing interagency agreements.

803.3 Up-Grading Existing Drainage Facilities

(1) *Rehabilitation and Reconstruction Projects.* The hydraulic adequacy, as well as the structural adequacy of existing drainage facilities should be evaluated early in the project development process on pavement rehabilitation and highway reconstruction projects.

Repair or replacement of structurally deficient drainage structures and up-grading of hydraulically inadequate drainage facilities should, whenever practicable, be included in the work of the proposed project. A thorough investigation of upstream and downstream conditions is often required to reveal what adverse effects there may be with increasing the capacity or velocity of existing cross drainage.

A cooperative agreement should be negotiated when the proposed work includes the upgrading of an existing storm drain system under the jurisdiction of a local or other public agency.

(2) *Proposed Upstream Development.* Unless developers of land in the drainage basin upstream of existing State highways incorporate positive stormwater management practices, such as detention or retention storage basins within their improvement areas, the peak flow from stormwater runoff is nearly always increased. As a practical matter, minor increases in peak flow are usually not objectionable. However, uncontrolled upstream development or diversions can significantly increase the peak flow run-off causing the capacity of downstream drainage systems, including those within the State right of way, to be exceeded.

When reasonable solutions to potential drainage problems associated with such increased flows include the up-grading of drainage facilities within the State highway right-of-way, cooperative agreements with the responsible local agency should be negotiated. The local agency having permit authority has the responsibility for assessing liabilities and seeking commensurate funding for mitigation of run-off impacts from the developers. The local agency should not allow potentially harmful developments to proceed until all issues have been resolved. If it becomes apparent that the District, the local agency and the developer may not amiably reach agreement, the matter should be referred to Caltrans Legal Division before there is an impasse in the negotiations.

Caltrans financial participation in such drainage improvements must be based on the general rule stated in Index 803.2 Cooperative Agreements.

(3) *Hydraulically Inadequate Facilities.* Land use changes nearly always cause areas to become less pervious and drainage basins to yield greater volumes and increase peak stormwater run-off flows. Even development of a small parcel of land within a drainage basin causes some increase in stormwater run-off. Individually the increase may be negligible. Collectively these incrementally small increases over time may cause the design capacity of an existing culvert to be exceeded.

The up-grading of this category of hydraulically inadequate drainage facilities may be partially or fully financed by Caltrans. Only if the benefit cost (b/c) ratio is equal to or greater than one is up-grading viable for normal Caltrans project funding. When the benefits to the Department and the traveling public do not justify increasing the capacity, up-grading may still be accomplished cooperatively with the local agency in accordance with the general rule for participation under Index 803.2 Cooperative Agreements.

Topic 804 - Floodplain Encroachments

804.1 Purpose

The purpose of these instructions is to provide uniform procedures and guidelines for Caltrans multi-disciplinary evaluation of proposed highway encroachments on floodplains.

804.2 Authority

Title 23, CFR, Part 650, Subpart A, prescribes FHWA's "...policies and procedures for the location and hydraulic design of highway encroachments on floodplains, ...". The CFR's may be found on-line at: http://www.access.gpo.gov/nara/cfr/cfr-table-search.html

804.3 Applicability

The guidance provided herein establishes Caltrans procedures whenever a floodplain encroachment is anticipated. Adherence to these procedures will also ensure compliance with applicable Federal regulations which apply to any Federally approved highway construction, reconstruction, rehabilitation, repair, or improvement project which affects the (100-year) base floodplain. Work outside the limits of the base floodplain should be reviewed to see if it affects the (100-year) base floodplain. The only exception is repairs made during or immediately following a disaster. The premise is that all Federal-aid projects be evaluated and that diligent efforts be made to:

- Avoid significant floodplain encroachments where practicable.
- Minimize the impact of highway actions that adversely affect the base floodplain.
- Be compatible with the National Flood Insurance Program (NFIP) of the Federal Emergency Management Agency (FEMA).

804.4 Definitions

The following definitions of terms are made for the purpose of uniform application in the documentation and preparation of floodplain evaluation reports. Refer to Title 23, CFR, Part 650, Section 650.105 for a complete list of definitions.

(1) *Base Flood.* The flood or tide having a 1 percent chance of being exceeded in any given year (100-year flood).

(2) *Base Floodplain.* The area subject to flooding by the base flood. Every watercourse (river, creek, swale, etc.) is subject to flooding and theoretically has a base floodplain.

(3) *Design Flood*. The peak discharge, volume if appropriate, stage or wave crest elevation of the flood associated with the probability of exceedance selected for the design of a highway encroachment. By definition, the highway will not be inundated from the stage of the design flood.

(4) *Encroachment.* An action within the limits of the base floodplain. Any construction activity (access road, building, fill slopes, bank or slope protection, etc.) within a base floodplain constitutes an encroachment.

(5) *Location Hydraulic Study.* A term from 23 CFR, Section 650.111 referring to the preliminary investigative study to be made of base floodplain encroachments by a proposed highway action. The extent of investigation and the discussion content in the required documentation of the "Location Hydraulic Study" is very site specific and need be no more than that which is commensurate with the risk(s) and impact(s) particular to the location under consideration. The information developed, documented (refer to Figure 804.7A) and retained in the project file is the suggested minimum necessary for compliance.

(6) *Natural and Beneficial Floodplain Values.* This shall include but is not limited to fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.

(7) *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.

(8) *Regulatory Floodway.* The floodplain area that is reserved in an open manner by Federal, State or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount (not to exceed 1 foot as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program).

804.5 Procedures

Floodplain evaluations are essentially an extension of the environmental assessment process and instructions contained in the Environmental Handbook and the Project Development Procedures Manual are to be followed. Early in the planning of a project it is necessary to first determine:

(a) If a proposed route alternative will encroach on a base floodplain (refer to Index 804.4 (2)) or,

(b) Where proposed construction on existing highway alignment encroaches on a base floodplain.

A Location Hydraulic Study is used to determine (a) and (b) above. Refer to Index 804.4 (4) and 804.7 (2)(b) for further discussion.

Where National Flood Insurance Program (NFIP) Maps and study reports are available, their use is mandatory in determining whether a highway location alternative will include an encroachment on the base floodplain. Three types of NFIP maps are published which, if available, may be obtained from the District Hydraulics Branch: Flood Hazard Boundary Map (FHBM), Flood Boundary and Floodway Map (FBFM), and Flood Insurance Rate Map (FIRM).

If NFIP Maps are not available, the District Hydraulics Engineer should develop hydrologic data and hydraulic information to estimate the limits of the 100-year base floodplain to determine whether a highway location alternative will include an encroachment.

Projects which involve proposed construction within a regulatory floodplain or floodway need to be analyzed to determine whether it may be necessary to obtain a map revision. A map revision is required when construction in the floodplain increases the base flood elevation (BFE) more than 1 foot. Not all new construction projects require a map revision.

804.6 Responsibilities

The District Project Engineer is generally the responsible party for initiating and coordinating the overall multi-disciplinary team activities of evaluation and documentation of floodplain impacts. Discussion of specific hydraulic and environmental aspects are required by 23 CFR 650, Subpart A. Preparing the project floodplain evaluation report and the summary for the environmental document or project report is normally the responsibility of the Environmental Planning Branch. The District Hydraulics Engineer will, as necessary, develop the hydrological and hydraulic information and provide technical assistance for assessing impacts of floodplain encroachments.

804.7 Preliminary Evaluation of Risks and Impacts for Environmental Document Phase

Virtually all proposed highway improvements that are considered as floodplain encroachments will be designed to have:

(a) No significant risks associated with implementation and,

(b) Negligible environmental impacts on the base floodplain.

(1) Risks. There will always be some potential for property damage and flooding that may affect public safety, associated with highway drainage design. In a majority of cases, a field review with a NFIP or USGS map and the application of good engineering judgment are all that is needed to determine if such risks are significant or acceptable. The detail of study and documentation shall be commensurate with the risk(s) or floodplain impact(s) and, in all cases, should be held to the minimum necessary to address 23 CFR 650.111.

(2) Impacts. The assessment of potential impacts on the floodplain environment will include:

(a) Impacts on natural and beneficial floodplain values.

(b) Support of probable incompatible floodplain development.

Except for the more environmentally sensitive projects, a single visit to the project site by the District Project Engineer, Hydraulics Engineer, and Environmental Planner, to assess and document the risks and environmental impacts associated with the proposed project is generally all that is necessary to obtain enough information for the "Location Hydraulic Study". Any reasonable adaptation of the technical information for “Location Hydraulic Study” form, Figure 804.7A, may be utilized to document and summarize the findings of the "Location Hydraulic Study" when the project is expected to be processed with a categorical exclusion. Items listed in 23 CFR 650.111 as follows must be addressed:

(a) National Flood Insurance Program (NFIP) maps or information developed by the highway agency, if NFIP maps are not available, shall be used to determine whether a highway location alternative will include an encroachment.

(b) Location studies shall include evaluation and discussion of the practicability of alternatives to any longitudinal encroachments.

(c) Location studies shall include discussion of the following items, commensurate with the significance of the risk or environmental impact, for all alternatives containing encroachments and for those actions which would support base floodplain development:

(1) The risks associated with implementation of the action,

(2) The impacts on natural and beneficial floodplain values,

(3) The support of probable incompatible floodplain develop-ment,

(4) The measures to minimize floodplain impacts associated with the action, and

(5) The measures to restore and preserve the natural and beneficial floodplain values impacted by the action.

(d) Location studies shall include evaluation and discussion of the practicability of alternatives to any significant encroachments or any support of incompatible floodplain development.

(e) The studies required by Sec. 650.111 (c) and (d) shall be summarized in environmental review documents prepared pursuant to 23 CFR part 771.

(f) Local, State, and Federal water resources and floodplain management agencies should be consulted to determine if the proposed highway action is consistent with existing watershed and floodplain management programs and to obtain current information on development and proposed actions in the affected watersheds.

Figure 804.7A is considered the suggested minimum hydraulic and engineering documentation for floodplain encroachments (bridge, culvert, channel change, slope protection, embankment, etc.). It is intended as a guide tool to help address the items listed in 23 CFR 650.111 and should be prepared jointly by the Project Engineer and Hydraulics Engineer. Since every location is unique, some of the questions may not apply, or additional considerations may need to be added.

For projects requiring an Environmental Impact Statement or Environmental Assessment (EIS/EA) or a finding of no significant impact (FONSI) with alternatives that have permanent features that encroach on the floodplain, a back-up report entitled Floodplain Evaluation is normally prepared by the District Environmental Branch. The technical requirements are typically developed jointly by the District Project Engineer and District Hydraulics Engineer. See Figure 804.7B for the Floodplain Evaluation Report Summary form that is used when an environmental document is to be prepared.

804.8 Design Standards

The design standards for highways encroaching on a floodplain are itemized in 23 CFR, Section 650.115. One requirement often overlooked is the need to assess the costs and risks associated with the overtopping flood for design alternatives in those instances where the overtopping flood exceeds the base flood. The content of design study information to be retained in the project file are described in 23 CFR, Section 650.117.

804.9 Coordination with the Local Community

The responsibility for enforcing National Flood Insurance Program (NFIP) regulations rests with the local community that is participating in the NFIP. It is the community who must submit proposals to Federal Emergency Management Agency (FEMA) for amendments to NFIP ordinances and maps in that community, or to demonstrate that an alternative floodway configuration meets NFIP requirements. However, this responsibility may be borne by the agency proposing to construct the highway crossing. Therefore, the highway agency should deal directly with the community and, through them, deal with FEMA. Determination of the status of a community's participation in the NFIP and review of applicable NFIP maps and study reports are, therefore, essential first steps in conducting location hydraulic studies and preparing environmental documents.

804.10 National Flood Insurance Program

The Flood Disaster Protection Act of 1973 (Pl 93-234, 87 Stat. 975) denies Federal financial assistance to flood prone communities that fail to qualify for flood insurance. The Act requires communities to adopt certain land use controls in order to qualify for flood insurance. These land use requirements could impose restrictions on the construction of highways in floodplains and regulatory floodplains in communities which have qualified for flood insurance.

The National Flood Insurance Act of 1968, as amended (42 U.S.C. 4001-4127) requires that communities adopt adequate land use and control measures to qualify for insurance. To implement this provision, the following Federal criteria contains requirements which may affect certain highways:

In riverine situations, when the Administrator of the Federal Insurance Administration has identified the flood prone area, the community must require that, until a floodway has been designated, no use, including land fill, be permitted within the floodplain area having special flood hazards for which base flood elevations have been provided, unless it has been demonstrated that the cumulative effect of the proposed use, when combined with all other existing and reasonably anticipated uses of similar nature, will not increase the water surface elevation of the 100-year flood more than 1 foot at any point within the community.

Figure 804.7A

Technical Information for Location Hydraulic Study

Dist. ________ Co. ________ Rte.________ P.M. _______________
EA __________________ Bridge No._________________
Floodplain Description ____
____
____

1. Description of Proposal (include any physical barriers i.e. concrete barriers, soundwalls, etc. and design elements to minimize floodplain impacts)
   ____
   ____
   ____

2. ADT: Current Projected

3. Hydraulic Data: Base Flood Q_{100} =_______ CFS
   WSE_{100} = _______ The flood of record, if greater than Q_{100}:
   Q =_______ CFS WSE =_______
   Overtopping flood Q =______ CFS WSE = _______
   Are NFIP maps available? Yes_____ No_____
   Are NFIP studies available? Yes_____ No_____

		Yes	No
4.	Is the highway location alternative within a regulatory floodway?	______	______
5.	Attach map with flood limits outlined showing all buildings or other improvements within the base floodplain. Potential Q_{100} backwater damages:		
A.	Residences?	______	______
B.	Other Bldgs?	______	______
C.	Crops?	______	______
D.	Natural and beneficial Floodplain values?	______	______
6.	Type of Traffic:		
A.	Emergency supply or evacuation route?	______	______
B.	Emergency vehicle access?	______	______
C.	Practicable detour available?	______	______
D.	School bus or mail route?	______	______

7. Estimated duration of traffic interruption for 100-year event ______ hours.

8. Estimated value of Q_{100} flood damages (if any) - moderate risk level.
   A. Roadway $________
   B. Property $________
   Total $________

9. Assessment of Level of Risk
   Low ___ Moderate ___ High ___
   For High Risk projects, during design phase, additional Design Study Risk Analysis may be necessary to determine design alternative.

PREPARED BY:

____ ____________________
Signature - Dist. Hydraulic Engineer Date
(Item numbers 3, 4, 5, 7, 9)

Is there any longitudinal encroachment, significant encroachment, or any support of incompatible Floodplain development? No ____ Yes ____

If yes, provide evaluation and discussion of practicability of alternatives in accordance with 23 CFR 650.113

Information developed to comply with the Federal requirement for the Location Hydraulic Study Shall be retained in the project files.

____ ____________________
Signature - Dist. Project Engineer Date
(Item numbers 1, 2, 6, 8)

Figure 804.7B

Floodplain Evaluation Report Summary

Dist. _________ Co. _________ Rte._________ P.M_________________
Project No. ___________________ Bridge No._________________
Limit

____

____

Floodplain Description

____

____

		Yes	No
1.	Is the proposed action a longitudinal encroachment of the base floodplain?	_____	_____
2.	Are the risks associated with the implementation of the proposed action significant?	_____	_____
3.	Will the proposed action support probable incompatible floodplain development?	_____	_____
4.	Are there any significant impacts on natural and beneficial floodplain values?	_____	_____
5.	Routine construction procedures are required to minimize impacts on the floodplain. Are there any special mitigation measures necessary to minimize impacts or restore and preserve natural and beneficial floodplain values? If yes, explain.	_____	_____
6.	Does the proposed action constitute a significant floodplain encroachment as defined in 23 CFR, Section 650.105(q).	_____	_____
7.	Are Location Hydraulic Studies that document the above answers on file? If not explain.	_____	_____

PREPARED BY:

____ _____________________
Signature - Dist. Hydraulic Engineer Date

____ _____________________
Signature - Dist. Environmental Branch Chief Date

____ _____________________
Signature - Dist. Project Engineer Date

- After the floodplain area having special flood hazards has been identified and the water surface elevation for the 100-year flood and floodway data have been provided, the community must designate a floodway which will convey the 100-year flood without increasing the water surface elevation of the flood more than 1 foot at any point and prohibit, within the designated floodway, fill, encroachments and new construction and substantial improvements of existing structures which would result in any increase in flood heights within the community during the occurrence of the 100-year flood discharge.
- The participating cities and/or counties agree to regulate new development in the designated floodplain and floodway through regulations adopted in a floodplain ordinance. The ordinance requires that development in the designated floodplain be consistent with the intent, standards and criteria set by the National Flood Insurance Program.

804.11 Coordination with FEMA

There should be Caltrans coordination with FEMA in situations where administrative determinations are needed involving a regulatory floodway or where flood risks in NFIP communities are significantly impacted. The circumstances which would ordinarily require coordination with FEMA include the following.

- When a proposed crossing encroaches on a regulatory floodway and, as such, would require an amendment to the floodway map.
- When a proposed crossing encroaches on a floodplain where a detailed study has been performed but no floodway designated and the maximum 1 foot increase in the base flood elevation would be exceeded.
- When a local community is expected to enter into the regular program within a reasonable period and detailed floodplain studies are under way.
- When a local community is participating in the emergency program and the base FEMA flood elevation in the vicinity of insurable buildings is increased by more than 1 foot. Where insurable buildings are not affected, it is sufficient to notify FEMA of changes to the base flood elevations as a result of highway construction.

The draft (EIS/EA) should indicate the NFIP status of affected communities, the encroachments anticipated and the need for floodway or floodplain ordinance amendments. If a determination by FEMA would influence the selection of an alternative, a commitment from FEMA should be obtained prior to the final environmental impact Statement (FEIS) or FONSI.

More information regarding FEMA can be found on-line at: http://www.fema.gov/nfip/.

FEMA has developed a comprehensive listing of all numerical models that are accepted for NFIP usage. These models can be accessed online at: http://www.fema.gov/mit/tsd/EN_modl.htm.

Topic 805 - Preliminary Plans

805.1 Required FHWA Approval

Current Federal policy requires the review and approval of plans for unusual structures. (See Indices 805.2 - 805.6) by FHWA. FHWA will no longer review and approve major structures (those with greater than 125,000 square feet of deck area) or pumping plants with greater than 20 CFS design discharge. Submittal of plans for unusual structures for review applies only to new construction on the Interstate system. The responsibility for the oversight of unusual structures on other Federal-aid and non-Federal-aid highways will be assumed by the state.

Federal review and approval may take place at either their Division Office or FHWA Headquarters in Washington, D.C. Early submission of necessary data is critical in order to receive a timely approval.

805.2 Bridge Preliminary Report

A Bridge Preliminary Report will be prepared by Structures Design, in the Division of Engineering Services and submitted to the California FHWA Division Office in Sacramento for approval of unusual bridges and structures.

An unusual bridge involves difficult or unique foundation problems, new or complex designs involving unique design or operational features, longer than normal spans or bridges for which the design procedures depart from current acceptable practice. Examples include cable stayed, suspension, arch, segmental concrete bridges, trusses and other bridges which deviate from AASHTO Standard Specifications or Guide Specifications for Highway Bridges, bridges requiring abnormal dynamic analysis for seismic design, bridges designed using a three-dimensional computer analysis, bridges with spans exceeding 500 feet, and bridges which include ultra high strength concrete or steel.

805.3 Storm Drain Systems

The District will submit preliminary plans and hydraulic data for unusual storm drain systems to the California FHWA Division Office in Sacramento for storm drain systems that carry more than 200 CFS or have an accumulated surface detention storage system of more than five acre-feet.

805.4 Unusual Hydraulic Structures

The District will submit preliminary plans and hydraulic data for unusual hydraulic structures to the California FHWA Office in Sacramento. For projects on the interstate system, FHWA Headquarters Office of Bridge Technology approval is required for hydraulic structures involving unusual stream stability countermeasures or unique design techniques. The Division of Engineering Services will submit preliminary plans and hydraulic data to the California FHWA Division Office in Sacramento for unusual structures such as tunnels, complex or unique geotechnical structures and complex or unique hydraulic structures.

805.5 Levees and Dams Formed by Highway Fills

The District will submit preliminary plans and other supportive data to the California FHWA Division Office in Sacramento for approval of:

(a) Highway fills which will function as a levee and serve the purpose of reducing the flooding of adjacent areas.

(b) Dams formed by highway fills which will permanently impound water more than 25 feet in depth or 50 acre-feet in volume. See Index 829.9 Dams, for legal definition of a dam and regulations relative to approval by the California Department of Water Resources.

805.6 Geotechnical

The District shall submit preliminary plans and technical data for major or unusual geotechnical features to the California FHWA Division Office for approval. Major geotechnical features include unusually deep cuts or high fills where the site geology is potentially unstable, landslide corrections, and large retaining walls (cantilever, permanent ground anchor, and soil reinforcement). FHWA Headquarters Bridge Division approval is required for unusual geotechnical features, such as new or complex retaining wall systems or ground improvement systems.

805.7 Data Provided by the District

The following items of supportive information must be provided with requests for FHWA approval:

(a) Preliminary plans and profiles:

- Approach layouts.
- Drainage plans.

(b) Hydraulic design studies:

- Design Q and frequency.
- Hydraulic grade lines.
- Inflow - Outflow hydrographs.
- Capacity of reservoirs or pump storage systems.
- Pump capacity.
- Stream velocities.
- Water surface profiles.
- Slope protection, toe and top elevations.

(c) Proposed specifications.

(d) Estimated cost.

(e) Foundation report:

- Embankment design for fills functioning as dams.

(f) Subsurface investigations.

(g) Coordination with Federal, state and local agencies.

(h) Other pertinent data.

The FHWA requires that three copies of supportive information be submitted to the California FHWA Division Office when approval by FHWA Headquarters Bridge Division is required. Four copies of supportive information are to be furnished to the Division of Engineering Services to prepare the FHWA approval requests for bridges.

Topic 806 - Definitions of Drainage Terms

806.1 Introduction

These definitions are for use with Sections 800 through 890 of this manual and the references cited. They are not necessarily definitions as established by case or statutory law.

806.2 Drainage Terms

Accretion. Outward growth of bank or shore by sedimentation. Increase or extension of boundaries of land by action of natural forces.

Action. Any highway construction, reconstruction, rehabilitation, repair, or improvement.

Aggradation. General and progressive raising of a stream bed by deposition of sediment. Modification of the earth's surface in the direction of uniformity of grade, or slope, by deposition as in a river bed.

Aggressive. Refers to the corrosive properties of soil and water.

Alluvial. Referring to deposits of silts, sands, gravels and similar detrital material which have been transported by running water.

Alluvium. Stream-borne materials deposited in and along a channel.

Apron. (1) A paved area (usually depressed) around a drainage inlet. (2) A floor or lining of concrete between wingwalls at the end of a culvert to prevent scour. (3) A lining of the bed of the channel upstream or downstream from a lined or restricted waterway. (4) A floor or lining of concrete, rock, etc., to protect a surface from erosion such as the pavement along the toe of bank protection.

Aqueduct. (1) A major conduit. (2) The entire transmission main for a municipal water supply which may consist of a succession of canals, pipes, tunnels, etc. (3) Any conduit for water; especially one for a large quantity of flowing water. (4) A structure for conveying a canal over a river or hollow.

Aquifer. Water-bearing geologic formations that permit the movement of ground water.

Armor. Artificial surfacing of bed, banks, shore or embankment to resist erosion or scour.

Arroyo. Waterway of an ephemeral stream deeply carved in rock or ancient alluvium.

Artesian Waters. Percolating waters confined below impermeable formations with sufficient pressure to spring or well up to the surface.

Articulated. Made flexible by hinging particularly of small rigid slabs adapted to revetment.

Avulsion. (1) A forcible separation; also, a part torn off. (2) The sudden removal of land from the estate of one man to that of another, as by a sudden change in a river, the property thus separated continuing in the original owner. (3) A sudden shift in location of channel.

Backing Layer. A layer of graded rock between rock riprap and underlying engineering fabric or filter layer to prevent extrusion of the soil or filter layer material through the riprap.

Backshore. The zone of the shore or beach lying between the foreshore and the coastline and acted upon by waves only during severe storms, especially when combined with exceptionally high water.

Backwater. An unnaturally high stage in stream caused by obstruction or confinement of flow, as by a dam, a bridge, or a culvert. Its measure is the excess of unnatural over natural stage, not the difference in stage upstream and downstream from its cause.

Baffle. Concrete or metal panels mounted in a series on the floor and/or wall of a culvert to increase boundary roughness and thereby reduce the average water velocity while increasing flow depth in the culvert.

Bank. The lateral boundary of a stream confining water flow. The bank on the left side of a channel looking downstream is called the left bank, etc.

Bankfull Stage. Stage at which a stream first overflows its natural banks into the floodplain. If the floodplain is absent or poorly defined, other indicators may identify bankfull. These include the height of depositional features, a change in vegetation, slope or topographic breaks along the bank, a change in the particle size of bank material, undercuts in the bank, and stain lines or the lower extent of lichens and moss on boulders. Corresponds to the stage at which channel maintenance is most effective, that is, the discharge at which the stream is moving sediment, forming or removing bars, forming or changing bends and meanders, and generally doing work that results in the average morphologic characteristics of channels. Generally applies to mature streams in more alluvial conditions rather than in mountainous conditions where the "bank" might be hundreds of feet above the incised channel. In incised channels, where the previous floodplain surface has become a terrace, the bankfull stage can be identified as the lowermost limit of establishing woody-riparian vegetation.

Bank Protection. Revetment, or other armor protecting a bank of a stream from erosion, includes devices used to deflect the forces of erosion away from the bank.

Bar. An elongated deposit of alluvium within a channel or across its mouth.

Barrier. A low dam or rack built to control flow of debris.

Base Flood. The flood or tide having a 1 percent chance of being exceeded in any given year (100-year flood). The "base flood" is commonly used as the "standard flood" in Federal flood insurance studies. (see Regulatory Flood).

Base Floodplain. The area subject to flooding by the base flood.

Basin. (1) The surface of the area tributary to a stream or lake. (2) Space above or below ground capable of retaining or detaining water or debris.

Bay. An indentation of bank or shore, including erosional cuts and slipouts, not necessarily large.

Beach. The zone of sedimentary material that extends landward from the low water line to the place where there is marked change in material or form, or to the line of permanent vegetation (usually the effective limit of storm waves). The seaward limit of a beach, unless otherwise specified, is the mean low water line. A beach includes foreshore and backshore.

Bed. The earth below any body of water, limited laterally by bank or shore.

Bedding. The foundation under a drainage structure.

Bed Load. Sediment that moves by rolling, sliding, or skipping along the bed and is essentially in contact with the stream bed.

Berm. (1) A bench or terrace between two slopes. (2) A nearly horizontal part of the beach or backshore formed at the high water line by waves depositing material. Some beaches have no berms, others have one or several.

Block. Precast prismatic unit for riprap structure.

Bluff. A high, steep bank composed of erodible materials.

Boil. Turbulent break in a water surface by upwelling.

Boom. Floating log or similar element designed to dampen surface waves or control the movement of drift.

Bore. A transient solitary wave in a narrow or converging channel advancing with a steep turbulent front; product of flash floods or incoming tides.

Boulder. Largest rock transported by a stream or rolled in the surf; typically heavier than 25 pounds and larger than 8 inches in diameter.

Braided Stream. A stream in which flow is divided at normal stage by small islands. This type of stream has the aspect of a single large channel with which there are subordinate channels.

Breaker. A collapsing wave meeting a shore, reef, sandbar, or rock.

Breakwater. A fixed or floating structure that protects a shore area, harbor, anchorage, or basin from intercepting waves.

Bulkhead. A steep or vertical structure placed on a bank, bluff, or embankment to retain or prevent sliding of the land and protect the inland area from damage.

Bulking. The increase in volume of flow due to air entrainment, debris, bedload, or sediment in suspension.

Buoyancy. Uplift force on a submerged body equal to the mass of water displaced times the acceleration of gravity.

Camber. An upward adjustment of the profile of a drainage facility under a heavy loading (usually a high embankment) and poor soil conditions, so that as the drainage facility settles it approaches the design profile.

Canal. An artificial open channel.

Canyon. A large deep valley; also the submarine counterpart.

Cap. Top layer of stone protective works.

Capacity. The effective carrying ability of a drainage structure. Generally measured in cubic feet per second.

Capillarity. The attraction between water and soil particles which cause water to move in any direction through the soil mass regardless of gravitational forces.

Capillary Water. Water which clings to soil particles by capillary action. It is normally associated with fine sand, silt, or clay, but not normally with coarse sand and gravel.

Catch Basin. A drainage structure which collects water. May be either a structure where water enters from the side or through a grating.

Causeway. A raised embankment or trestle over swamp or overflow areas.

Cavitation. Erosion by suction, especially in the partial vacuum of a diverging jet.

Celerity. Velocity of a moving wave, as distinguished from velocity of particles oscillating in the wave.

Channel. An open conduit either naturally or artificially created which periodically or continuously contains moving water, or which forms a connecting link between two bodies of water. River, creek, run, branch, anabranch, and tributary are some of the terms used to describe natural channels. Natural channels may be single or braided (see Braided Stream). Canal and "floodway" are some of the terms used to describe artificial channels.

Check. A sill or weir in a channel to control stage or velocity.

Check Dam. A small dam generally placed in steep ditches for the purpose of reducing the velocity in the ditch.

Cienega. A swamp formed by water rising to the surface at a fault.

Cleanout. An access opening to a roadway drainage system. Usually consists of a manhole shaft, a special chamber or opening into a shallow culvert or drain.

Cliff. A high, steep face of rock; a precipice.

Cloudburst. Rain storm of great intensity usually over a small area for a short duration.

Coast. (1) The strip of land, of indefinite width (up to several miles), that extends from the shoreline inland to the first major change in terrain features. (2) As a combining form, "upcoast" is northerly and "downcoast" is southerly.

Cobble. Rock smaller than a boulder and larger than gravel; typically 1 pound to 25 pounds, or 3 inches to 8 inches in diameter.

Coefficient of Runoff. Percentage of gross rainfall which appears as runoff.

Composite Hydrograph. A plot of mean daily discharges for a number of years of record on a single year time base for the purpose of showing the occurrence of high and low flows.

Concentrated Flow. Flowing water that has been accumulated into a single fairly narrow stream.

Concentration. In addition to its general sense, means the unnatural collection or convergence of waters so as to discharge in a narrower width, and at greater depth or velocity.

Conduit. Any pipe, arch, box or drain tile through which water is conveyed.

Cone. Physiographic form of sediment deposit washed from a gorge channel onto an open plain; a debris cone, also called an alluvial fan.

Confluence. A junction of streams.

Constriction. An obstruction narrowing a waterway.

Contraction. The reduction in cross sectional area of flow.

Control. (1) A section or reach of an open conduit or stream channel which maintains a stable relationship between stage and discharge. (2) For flood, erosion, debris, etc., remedial means or procedure restricting damage to a tolerable level.

Conveyance. A measure of the water carrying capacity of a stream or channel.

Core. Central zone of dike, levee, rock groin, jetty, etc.

Corrasion. Erosion or scour by abrasion in flowing water.

Corrosion. Erosion by chemical action.

Cradle. (1) A concrete base generally constructed to fit the shape of a structure which is to be forced through earthen material by a jacking operation. The cradle is constructed to line and grade. (2) Wood support for rigid culverts on yielding embankment subgrade. Then the pipe rides on the cradle as it is worked through the given material by jacking and tunneling methods. Also serves as bedding for pipes in trenches in special conditions.

Creek. A small stream, usually active.

Crest. (1) Peak of a wave or a flood. (2) Top of a levee, dam, weir, spillway or other water barrier or control.

Crib. An open-frame structure loaded with earth or stone ballast to act as a baffle in bank protection.

Critical Depth. (Depth at which specific energy is a minimum) - The depth of water in a conduit at which under certain other conditions the maximum flow will occur. These other conditions are the conduit is on the critical slope with the water flowing at its critical velocity and there is an adequate supply of water. The depth of water flowing in an open channel or a conduit partially filled, for which the velocity head equals one-half the hydraulic mean depth.

Critical Flow. That flow in open channels at which the energy content of the fluid is at a minimum. Also, that flow which has a Froude number of one.

Critical Slope. That slope at which the maximum flow will occur at the minimum velocity. The slope or grade that is exactly equal to the loss of head per foot resulting from flow at a depth that will give uniform flow at critical depth; the slope of a conduit which will produce critical flow.

Critical Velocity. Mean velocity of flow when flow is at critical depth.

Culvert. A closed conduit which allows water to pass under a highway. The following three conditions constitute a culvert;

1. Single Barrel - span measured along centerline of road 20 feet or less.
2. Multi-Barrels - total of the individual spans measured along centerline of road is 20 feet or less.
3. Multi-Barrels - total of the individual spans measured along centerline of road is 20 feet or greater, but the distance between individual culverts is more than one-half the culvert diameter.

Current. Flow of water, both as a phenomenon and as a vector. Usually qualified by adjectives like downward, littoral, tidal, etc. to show relation to a pattern of movement.

Current Meter. An instrument for measuring the velocity of a current. It is usually operated by a wheel equipped with vanes or cups which is rotated by the action of the impinging current. An indicating or recording device is provided to indicate the speed of rotation which is correlated with the velocity of the current.

Cutoff Wall. A wall at the end of a drainage structure, the top of which is an integral part of the drainage structure. This wall is usually buried and its function is to prevent undermining of the drainage structure if the natural material at the outlet of the structure is dug out by the water discharging from the end of the structure. Cutoff walls are sometimes used at the upstream end of a structure when there is a possibility of erosion at this point.

Debris. Any material including floating woody materials and other trash, suspended sediment, or bed load moved by a flowing stream.

Debris Barrier. A deflector placed at the entrance of a culvert upstream, which tends to deflect heavy floating debris or boulders away from the culvert entrance during high-velocity flow.

Debris Basin. Any area upstream from a drainage structure utilized for the purpose of retaining debris in order to prevent clogging of drainage structures downstream.

Debris Rack. A straight barrier placed across the stream channel which tends to separate light and medium floating debris from stream flow and prevent the debris from reaching the culvert entrance.

Degradation. General and progressive lowering of the longitudinal profile of a channel by erosion.

Delta. System of channels thru an alluvial plain at the mouth of a stream.

Deposit. An earth mass of particles settled or stranded from moving water or wind.

Depth. Vertical distance, (1) from surface to bed of a body of water. (2) From crest or crown to invert of a conduit.

Design Capacity. The size required of a drainage facility which allows it to pass the design discharge without detrimental impacts.

Design Channel Capacity. Expressed as a rate of flow, usually in cubic feet per second, it is the level to which a facility is designed. Based upon slope, geometry, flow regime, frictional coefficients, etc., it is the sizing of a drainage facility which allows it to pass the design discharge. Freeboard or other safety factors which are added to the final facility dimensions are not a part of the design capacity.

Design Discharge. The quantity of flow that is expected at a certain point as a result of a design storm. Usually expressed as a rate of flow in cubic feet per second.

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. By definition, the highway will not be inundated by the design flood. In a FEMA floodplain, see 23 CFR, Part 650, Subpart A, for definitions of "overtopping flood" and "base flood."

Design Frequency. The recurrence interval for hydrologic events used for design purposes. As an example, a design frequency of 50 years means a storm of a magnitude that would be expected to recur on the average of every 50 years. (See Probability of Exceedance.)

Design High Water. The flood stage or tide crest elevation adopted for design of drainage and bank protection structures. (See Design Flood and High Water).

Design Storm. That particular storm which contributes runoff which the drainage facilities were designed to handle. This storm is selected for design on the basis of its probability of exceedance or average recurrence interval (See Probability of Exceedance.)

Detention Storage. Surface water moving over the land is in detention storage. Surface water allowed to temporarily accumulate in ponds, basins, reservoirs or other types of holding facility and which is ultimately returned to a watercourse or other drainage system as runoff is in detention storage. (See Retention Storage)

Detritus. Loose material such as; rock, sand, silt, and organic particles.

Dike. (1) Usually an earthen bank alongside and parallel with a river or open channel or an AC dike along the edge of a shoulder. (See Levee) (2) An AC dike along the edge of a shoulder.

Dike, Finger. Relatively short embankments constructed normal to a larger embankment, such as an approach fill to a bridge. Their purpose is to impede flow and direct it away from the major embankment.

Dike, Toe. Embankment constructed to prevent lateral flow from scouring the corner of the downstream side of an abutment embankment. Sometimes referred to as training dikes.

Dike, Training. Embankments constructed to provide a transition from the natural stream channel or floodplain, both to and from a constricting bridge crossing.

Discharge. A volume of water flowing out of a drainage structure or facility. Measured in cubic feet per second.

Dissipate. Expend or scatter harmlessly, as of energy of moving water.

Ditch. Small artificial channel, usually unlined.

Diversion. (1) The change in character, location, direction, or quantity of flow of a natural drainage course (a deflection of flood water is not a diversion). (2) Draft of water from one channel to another. (3) Interception of runoff by works which discharge it thru unnatural channels.

D-Load (Cracking D-Load). A term used in expressing the strength of concrete pipe. The cracking D-load represents the test load required to produce a 0.01 inch crack for a length of 12 inches.

Downdrain. A prefabricated drainage facility assembled and installed in the field for the purpose of transporting water down steep slopes.

Downdrift. The direction of predominant movement of littoral materials.

Drain. Conduit intercepting and discharging surplus ground or surface water.

Drainage. (1) The process of removing surplus ground or surface water by artificial means. (2) The system by which the waters of an area are removed. (3) The area from which waters are drained; a drainage basin.

Drainage Area (Drainage Basin) (Basin). That portion of the earth's surface upon which falling precipitation flows to a given location. With respect to a highway, this location may be either a culvert, the farthest point of a channel, or an inlet to a roadway drainage system.

Drainage Course. Any path along which water flows when acted upon by gravitational forces.

Drainage Divide. The rim of a drainage basin. A series of high points from which water flows in two directions, to the basin and away from the basin.

Drainage Easement (See Easement).

Drainage System. Usually a system of underground conduits and collector structures which flow to a single point of discharge.

Drawdown. The difference in elevation between the water surface elevation at a constriction in a stream or conduit and the elevation that would exist if the constriction were absent. Drawdown also occurs at changes from mild to steep channel slopes and weirs or vertical spillways.

Drift. (1) Floating or non-mineral burden of a stream. (2) Deviation from a normal course in a cross current, as in littoral drift.

Drop. Controlled fall in a stream to dissipate energy.

Dry Weather Flows. A small amount of water which flows almost continually due to lawn watering, irrigation or springs.

Dune. A sand wave of approximately triangular cross section (in a vertical plane in the direction of flow) formed by moving water or wind, with gentle upstream slope and steep downstream slope and deposition on the downstream slope.

Easement. Right to use the land of others.

Ebb. Falling stage or outward flow, especially of tides.

Eddy. Rotational flow around a vertical axis.

Eddy Loss. The energy lost (converted into heat) by swirls, eddies, and impact, as distinguished from friction loss.

Embankment. Earth structure above natural ground.

Embayment. Indentation of bank or shore, particularly by progressive erosion.

Encroachment. Extending beyond the original, or customary limits, such as by occupancy of the river and/or floodplain by earth fill embankment.

Endwall. A wall placed at the end of a culvert. It may serve three purposes; (1), to hold the embankment away from the pipe and prevent sloughing into the pipe outlet channel; (2), to provide a wall which will prevent erosion of the roadway fill; and (3), to prevent flotation of the pipe.

Energy. Potential or kinetic, the latter being expressed in the same unit (feet) as the former.

Energy Dissipator. A structure for the purpose of slowing the flow of water and reducing the erosive forces present in any rapidly flowing body of water.

Energy Grade Line. The line which represents the total energy gradient along the channel. It is established by adding together the potential energy expressed as the water surface elevation referenced to a datum and the kinetic energy (usually expressed as velocity head) at points along the stream bed or channel floor.

Energy Head. The elevation of the hydraulic grade line at any section plus the velocity head of the mean velocity of the water in that section.

Entrance. The upstream approach transition to a constricted waterway.

Entrance Head. The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head.

Entrance Loss. The head lost in eddies and friction at the inlet to a conduit or structure.

Ephemeral. Of brief duration, as the flow of a stream in an arid region.

Equalizer. A drainage structure similar to a culvert but different in that it is not intended to pass a design flow in a given direction. Instead it is often placed level so as to permit passage of water in either direction. It is used where there is no place for the water to go. Its purpose is to maintain the same water surface elevation on both sides of the highway embankment.

Erosion. The wearing away of natural (earth) and unnatural (embankment, slope protection, structure, etc.) surfaces by the action of natural forces, particularly moving water and materials carried by it. In the case of drainage terminology, this term generally refers to the wearing away of the earth's surface by flowing water.

Erosion and *Scour*. The cutting or wearing away by the forces of water of the banks and bed of a channel in horizontal and vertical directions, respectively.

Erosion and *Accretion*. Loss and gain of land, respectively, by the gradual action of a stream in shifting its channel by cutting one bank while it builds on the opposite bank. Property is lost by erosion and gained by accretion but not by *avulsion* when the shift from one channel to another is sudden. Property is gained by *reliction* when a lake recedes.

Estuary. That portion of a river channel occupied at times or in part by both sea and river flow in appreciable quantities. The water usually has brackish characteristics.

Evaporation. A process whereby water as a liquid is changed into water vapor, typically through heat supplied from the sun.

Face. The outer layer of slope revetment.

Fan. A portion of a cone, but sometimes used to emphasize definition of radial channels. Also reference to spreading out of water or soils associated with waters leaving a confined channel (e.g., alluvial fan).

Fetch. The unobstructed distance across open water through which wind acts to generate waves.

Filter. A porous article or mass (as of fabric or even-graded mineral aggregate) through which water will freely pass, but which will block the passage of soil particles.

Filter Fabric (RSP fabric). An engineering fabric (geotextile) placed between the backfill and supporting or underlying soil through which water will pass and soil particles are retained.

Filter Layer. A layer of even-graded rock between rock riprap and underlying soil to prevent extrusion of the soil thru riprap.

Flap Gate. This is a form of valve that is designed so that a minimum force is required to push it open but when a greater water pressure is present on the outside of the valve, it remains shut so as to prevent water from flowing in the wrong direction. Construction is simple with a metal cover hanging from an overhead rod or pinion at the end of a culvert or drain.

Flood Frequency. Also referred to as exceedance interval, recurrence interval or return period; the average time interval between actual occurrences of a hydrological event of a given or greater magnitude; the percent chance of occurrence is the reciprocal of flood frequency, e.g., a 2 percent chance of occurrence is the reciprocal statement of a 50-year flood. (See Probability of Exceedance.)

Floodplain. Normally dry land areas subject to periodic temporary inundation by stream flow or tidal overflow. Land formed by deposition of sediment by water; alluvial land.

Floodplain Encroachment. An action within the limits of the base floodplain.

Flood Plane. The position occupied by the water surface of a stream during a particular flood. Also, loosely, the elevation of the water surface at various points along the stream during a particular flood.

Floodproof. To design and construct individual buildings, facilities, and their sites to protect against structural failure, to keep water out or reduce the effects of water entry.

Flood Stage. The elevation at which overflow of the natural banks of a stream begins to cause damage in the reach in which the elevation is measured. The elevation of the lowest bank of the reach. The term "lowest bank" is, however, not to be taken to mean an unusually low place or break in the natural bank through which the water inundates an unimportant and small area.

Flood Waters. Former stream waters which have escaped from a watercourse (and its overflow channel) and flow or stand over adjoining lands. They remain as such until they disappear from the surface by infiltration, evaporation, or return to a natural watercourse. They do not become surface waters by mingling with such waters, nor stream waters by eroding a temporary channel.

Flow. A term used to define the movement of water, silt, sand, etc.; discharge; total quantity carried by a stream.

Flow Line. A term used to describe the line connecting the low points in a watercourse.

Flow Regime. The system or order characteristic of streamflow with respect to velocity, depth, and specific energy.

Flow, steady. Flow at constant discharge.

Flow, unsteady. Flow on rising or falling stages.

Flow, varied. Flow in a channel with variable section.

Foreshore. The part of the shore lying between the ordinary high water mark or upper limit of wave wash traversed by the runup and return of waves and the water's edge at the low water.

Freeboard. (1) The vertical distance between the water surface elevation usually corresponding to the design flow and a point of interest such as a bridge beam, levee top or specific location on the roadway grade. (2) The distance between the normal operating level and the top of the sides of an open conduit; the crest of a dam, etc., designed to allow for wave action, superelevation, floating debris, or any other condition or emergency, without overtopping the structure. Freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccounted factors such as the accumulation of silt, trash, or aquatic growth in the channel; unforeseen embankment settlement, erratic hydrologic phenomena and variation of resistance or other coefficients from those assumed in design.

Free Outlet. A condition under which water discharges with no interference such as a pipe discharging into open air.

Free Water. Water which can move through the soil by force of gravity.

French Drain. A trench loosely backfilled with stones, the largest stones being placed in the bottom with the size of stones decreasing towards the top. The interstices between the stones serve as a passageway for water.

Friction. Energy-dissipating conflict among turbulent water particles disturbed by irregularities of channel surface.

Froude Number. A dimensionless expression of the ratio of inertia forces to gravity forces, used as an index to characterize the type of flow in a hydraulic structure in which gravity is the force producing motion and inertia is the resisting force. It is equal to a characteristic flow velocity (mean, surface, or maximum) of the system divided by the square root of the product of a characteristic dimension (as diameter of depth) and the gravity constant (acceleration due to gravity) all expressed in consistent units. $F_r = V/(gy)^{1/2}$

Gabion. A wire basket or cage filled with stone and placed as, or as part of, a bank-protection structure.

Gaging Station. A location on a stream where measurements of stage or discharge are customarily made. The location includes a reach of channel through which the flow is uniform, a control downstream from this reach and usually a small building to house the recording instruments.

Gorge. A narrow deep valley with steep or vertical banks.

Grade. Elevation of bed or invert of a channel.

Grade to Drain. A construction note often inserted on a plan for the purpose of directing the Contractor to slope a certain area in a specific direction, so that the surface waters will flow to a designated location.

Gradient (Slope). The rate of ascent or descent expressed as a percent or as a decimal as determined by the ratio of the change in elevation to the length.

Gradually Varied Flow. In this type of flow, changes in depth and velocity take place slowly over large distances, resistance to flow dominates and acceleration forces are neglected.

Grate. A framework of bars, usually cast iron or welded steel, used as a screen to cover the intake of a drainage inlet. See Standards Plans and Standard Specifications for requirements.

Ground Water. That water which is present under the earth's surface. Ground water is that situated below the surface of the land, irrespective of its source and transient status. Subterranean streams are flows of ground waters parallel to and adjoining stream waters, and usually determined to be integral parts of the visible streams.

Grouted. Bonded together with an inlay or overlay of cement mortar.

Guide Bank. An appendage to the highway embankment at or near a bridge abutment to guide the stream through the bridge opening.

Gulch. A relatively young, well-defined and sharply cut erosional channel.

Gully. Diminutive of gulch.

Head. Represents an available force equivalent to a certain depth of water. This is the motivating force in effecting the movement of water. The height of water above any point or plane of reference. Used also in various compound expressions, such as energy head, entrance head, friction head, static head, pressure head, lost head, etc.

Headcutting. Progressive scouring and degrading of a streambed at a relatively rapid rate in the upstream direction, usually characterized by one or a series of vertical falls.

High Water. Maximum flood stage of stream or lake; periodic crest stage of tide. Historic HW is stage recorded or otherwise known.

Hydraulic. Pertaining to water in motion and the mechanics of the motion.

Hydraulic Gradient. A line which represents the relative force available due to the potential energy available. This is a combination of energy due to the height of the water and the internal pressure. In any open channel, this line corresponds to the water surface. In a closed conduit, if several openings were placed along the top of the pipe and open tubes inserted, a line connecting the water surface in each of these tubes would represent the hydraulic grade line.

Hydraulic Jump (or Jump). Transition of flow from the rapid to the tranquil state. A varied flow phenomenon producing a rise in elevation of water surface. A sudden transition from supercritical flow to the complementary subcritical flow, conserving momentum and dissipating energy.

Hydraulic Mean Depth. The area of the flow cross section divided by the water surface width.

Hydraulic Radius. The cross sectional area of a stream of water divided by the length of that part of its periphery in contact with its containing conduit; the ratio of area to wetted perimeter.

Hydrograph. A graph showing stage, flow, velocity, or other property of water with respect to time.

Hydrographic. Pertaining to the measurement or study of bodies of water and associated terrain.

Hydrography. Water Surveys. The art of measuring, recording, and analyzing the flow of water; and of measuring and mapping watercourses, shore lines, and navigable waters.

Hydrologic. Pertaining to the cyclic phenomena of waters of the earth; successively as precipitation, runoff, storage and evaporation, and quantitatively as to distribution and concentration.

Hydrology. The science dealing with the occurrence and movement of water upon and beneath the land areas of the earth. Overlaps and includes portions of other sciences such as meteorology and geology. The particular branch of Hydrology that a design engineer is generally interested in is surface runoff which is the result of excess precipitation.

Hydrostatic. Pertaining to pressure by and within water due to gravitation acting thru depth.

Hyetograph. Graphical representation of rainfall intensity against time.

Impinge. To strike and attack directly, as in curvilinear flow where the current does not follow the curve but continues on tangent into the bank on the outside of bend in the channel.

Incised Channel. Those channels which have been cut relatively deep into underlying formations by natural processes. Characteristics include relatively straight alignment and high, steep banks such that overflow rarely occurs, if ever.

Infiltration. The passage of water through the soil surface into the ground.

Inlet Time. The time required for storm runoff to flow from the most remote point, in flow time, of a drainage area to the point where it enters a drain or culvert.

Inlet Transition. A specially shaped entrance to a box or pipe culvert. It is shaped in such a manner that in passing from one flow condition to another, the minimum turbulence or interference with flow is permitted.

Inundate. To cover with a flood.

Invert. The bottom of a drainage facility along which the lowest flows would pass.

Invert Paving. Generally applies to metal pipes where it is desirable to improve flow characteristics or prevent corrosion at low flows. The bottom portion of the pipe is paved with an asphaltic material, concrete, or air-blown mortar.

Inverted Siphon. A pipe for conducting water beneath a depressed place. A true inverted siphon is a culvert which has the middle portion at a lower elevation than either the inlet or the outlet and in which a vacuum is created at some point in the pipe. A sag culvert is similar, but the vacuum is not essential to its operation.

Isohyetal Line. A line drawn on a map or chart joining points that receive the same amount of precipitation.

Isohyetal Map. A map containing isohyetal lines and showing rainfall intensities.

Isovel. Line on a diagram of a channel connecting points of equal velocity.

Jack (or Jack Straw). Bank protection element consisting of wire or cable strung on three mutually perpendicular struts connected at their centers.

Jacking Operations. A means of constructing a pipeline under a highway without open excavation. A cutting edge is placed on the first section of pipe and the pipe is forced ahead by hydraulic jacks. As the leading edge pushes ahead, the material inside the pipe is dug out and transported outside the pipe for disposal.

Jam. Wedged collection of drift in a constriction of a channel, such as a gorge or a bridge opening.

Jet. An effluent stream from a restricted channel, including a fast current through a slower stream.

Jetty. An elongated, artificial obstruction projecting into a stream or the sea from bank or shore to control shoaling and scour by deflection of strength of currents and waves.

Jump. Sudden transition from supercritical flow to the complementary subcritical flow, conserving momentum and dissipating energy; the hydraulic jump.

Kolk. Rotational flow about a horizontal axis, induced by a reef and breaking the surface in a boil.

Lake. A water filled basin with restricted or no outlet. Includes reservoirs, tidal ponds and playas.

Lag. Variously defined as time from beginning (or center of mass) of rainfall to peak (or center of mass) of runoff.

Laminar Flow. That type of flow in which each particle moves in a direction parallel to every other particle and in which the head loss is approximately proportional to the velocity (as opposed to turbulent flow).

Lateral. In a roadway drainage system, a drainage conduit transporting water from inlet points to the main drain trunk line.

Levee. An embankment on or along the bank of a stream or lake to protect outer lowlands from inundation. (See Dike)

Lining. Protective cover of the perimeter of a channel.

Littoral. Pertaining to or along the shore, particularly to describe currents, deposits, and drift.

Littoral Drift. The sedimentary material (sand) moved along the shoreline under the influence of waves and currents.

Littoral Transport. The movement of littoral drift along the shoreline by waves and currents. Includes movement parallel (longshore transport) and perpendicular (on-offshore transport) to the shore.

Local Depression. A low area in the pavement or in the gutter established for the special purpose of collecting surface waters on a street and directing these waters into a drainage inlet.

Longshore. Parallel to and near the shoreline.

Marginal. Within a borderland area; more general and extensive than riparian.

Marsh. An area of soft, wet, or periodically submerged land, generally treeless and usually characterized by grasses and other low vegetation.

Mature. Classification for streams which have established flat gradients not subject to further scour.

Maximum Historical Flood. The maximum flood that has been recorded or experienced at any particular highway location.

Mean Annual Flood. The flood discharge with a recurrence interval of 2.33 years.

Mean Depth. For a stream at any stage, the wetted normal section divided by the surface width. Hydraulic mean depth.

Meander. In connection with streams, a winding channel usually in an erodible, alluvial valley. A reverse or S-shaped curve or series of curves formed by erosion of the concave bank, especially at the downstream end, characterized by curved flow and alternating shoals and bank erosions. Meandering is a stage in the migratory movement of the channel, as a whole, down the valley.

Meander Plug (Clay Plug). Deposits of cohesive materials in old channel bendways. These plugs are sufficiently resistant to erosion to serve as essentially semi-permanent geological controls to advancing channel migrations.

Meander Scroll. Evidence of historical meander patterns in the form of lines visible on the inside of meander bends (particularly on aerial photographs) which resemble a spiral or convoluted form in ornamental design. These lines are concentric and regular forms in high sinuosity channels and are largely absent in poorly developed braided channels.

Mesh. Woven wire or other filaments used alone as revetment, or as retainer or container of masses of gravel or cobble.

Mud Flow. A well-mixed mass of water and alluvium which, because of its high viscosity, and low fluidity as compared with water, moves at a much slower rate, usually piling up and spreading out like a sheet of wet mortar or concrete.

Natural and Beneficial Floodplain Values. Includes but are not limited to fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, aquaculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.

Natural Channel Capacity. The maximum rate of flow in cubic feet per second that can pass through a channel without overflowing the banks

Navigable Waters. Those stream waters lawfully declared or actually used as such. Navigable Waters of the State of California are those declared by Statute. Navigable Waters of the United States are those determined by the Corps of Engineers or the U.S. Coast Guard to be so used in interstate or international commerce. Other streams have been held navigable by courts under the common law that navigability in fact is navigability in law.

Negative Projecting Conduits. A structure installed in a trench with the top below the top of trench, then covered with backfill and embankment. See Positive Projecting Conduit

Nonuniform Flow. A flow in which the velocities vary from point to point along the stream or conduit, due to variations in cross section, slope, etc.

Normal Depth. The depth at which flow is steady and hydraulic characteristics are uniform.

Normal Water Surface (Natural Water Surface). The free surface associated with flow in natural streams.

"n" Value. The roughness coefficient in the Manning formula for determination of the discharge coefficient in the Chezy formula,

$$V = C(RS)^{1/2}\text{, where } C = \left(\frac{1.49}{n}\right)R^{1/6}$$

Nourishment. The process of replenishing a beach. It may be brought about naturally, by accretion due to the longshore transport, or artificially, by the deposition of dredged materials.

Off-Site Drainage. The handling of that water which originates outside the highway right of way.

On-Site Drainage. The handling of that water which originates inside the highway right of way.

Open Channel. Any conveyance in which water flows with a free surface.

Ordinary High Water Mark. The line on the shore established by the fluctuation of water and physically indicated on the bank (1.5 $\pm$ years return period)

Outfall. Discharge or point of discharge of a culvert or other closed conduit.

Outwash. Debris transported from a restricted channel to an unrestricted area where it is deposited to form an alluvial or debris cone or fan.

Overflow. Discharge of a stream outside its banks; the parallel channels carrying such discharge.

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.

Peak Flow. Maximum momentary stage or discharge of a stream in flood. Design Discharge.

Pebble. Stone 0.5 inch to 3-inch in diameter, including coarse gravel and small cobble.

Perched Water. Ground water located above the level of the water table and separated from it by a zone of impermeable material.

Percolating Waters. Waters which have infiltrated the surface of the land and move slowly downward and outward through devious channels (aquifers) unrelated to stream waters, until they reach an underground lake or regain and spring from the land surface at a lower point.

Permeability. The property of soils which permits the passage of any fluid. Permeability depends on grain size, void ratio, shape and arrangement of pores.

Permeable. Open to the passage of fluids, as for (1) pervious soils and (2) bank-protection structures.

Physiographic Region. A geographic area whose pattern of landforms differ significantly from that of adjacent regions.

Pier. Vertical support of a structure standing in a stream or other body of water. Used in a general sense to include bents and abutments.

Pile. A long, heavy timber or section of concrete or metal that is driven or jetted into the earth or bottom of a water body to serve as a structural support or protection.

Piping. The action of water passing through or under an embankment and carrying some of the finer material with it to the surface at the downstream face.

Plunge. Flow with a strong downward component, as in outfall drops, overbank falls, and surf attack on a beach.

Point of Concentration. That point at which the water flowing from a given drainage area concentrates. With reference to a highway, this would generally be either a culvert entrance or some point in a roadway drainage system.

Poised Stream. A term used by river engineers applying to a stream that over a period of time is neither degrading or aggrading its channel, and is nearly in equilibrium as to sediment transport and supply.

Positive Projecting Conduit. A structure installed in shallow trench with the top of the conduit projecting above the top of the trench and then covered with embankment. See Negative Projecting Conduit.

Potamology. The hydrology of streams.

Practicable. Capable of being done within reasonable natural, social, and economic constraints.

Precipitation. Discharge of atmospheric moisture as rain, snow or hail, measured in depth of fall or in terms of intensity of fall in unit time.

Prescriptive Rights. The operation of the law whereby rights may be established by long exercise of their corresponding powers or extinguished by prolonged failure to exercise such powers.

Preserve. To avoid modification to the functions of the natural floodplain environment or to maintain it, as closely as practicable, in its natural state.

Probability. The chance of occurrence or recurrence of a specified event within a unit of time, commonly expressed in 3 ways. Thus a 10-year flood has a chance of 0.1 per year and is also called a 10 percent-chance flood.

Probability of Exceedance. The statistical probability, expressed as a percentage, of a hydrologic event occurring or being exceeded in any given year. The probability (p) of a storm or flood is the reciprocal of the average recurrence interval (N).

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.

Pumping Plant. A complete pumping installation including a storage box, pump or pumps, standby pumps, connecting pipes, electrical equipment, pumphouse and outlet chamber.

Rack. An open upright structure, such as a debris rack.

Rainfall. Point Precipitation: That which registers at a single gauge. Area Precipitation: Adjusted point rainfall for area size.

Rainwash. The creep of soil lubricated by rain.

Range. Difference between extremes, as for stream or tide stage.

Rapidly Varied Flow. In this type of flow, changes in depth and velocity take place over short distances, acceleration forces dominate, and energy loss due to friction is minor.

Rapids. Swift turbulent flow in a rough steep reach.

Reach. The length of a channel uniform with respect to discharge, depth, area, and slope. More generally, any length of a river or drainage course.

Recession. Retreat of shore or bank by progressive erosion.

Reef. Generally, any solid projection from the bed of a stream or other body of water.

Regime. The system or order characteristic of a stream; its behavior with respect to velocity and volume, form of and changes in channel, capacity to transport sediment, amount of material supplied for transportation, etc.

Regimen. The characteristic behavior of a stream during ordinary cycles of flow.

Regulatory Floodway. The open floodplain area that is reserved in by Federal, State, or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount (not to exceed 1 foot as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program (NFIP)).

Reliction. Pertaining to being left behind. For example: that area of land is left behind by reliction when the water surface of a lake is lowered.

Repose. The stable slope of a bank or embankment, expressed as an angle or the ratio of horizontal to vertical projection.

Restore. To reestablish a setting or environment in which the functions of the natural and beneficial floodplain values adversely impacted by the highway agency can continue to operate.

Restriction. Artificial or natural control against widening of a channel, with or without construction.

Retard. Bank-protection structure designed to check the riparian velocity and induce silting or accretion.

Retarding Basin. Either a natural or man made basin with the specific function of delaying the flow of water from one point to another. This tends to increase the time that it takes all the water falling on the extremities of the drainage basin to reach a common point, resulting in a reduced peak flow at that point.

Retention Storage. Water which accumulates and ponds in natural or excavated depressions in the soil surface with no possibility for escape as runoff. (See Detention Storage)

Retrogression. Reversal of stream grading; i.e., aggradation after degradation, or vice versa.

Revetment. Bank protection to prevent erosion.

Riparian. Pertaining to the banks of a stream.

Riprap. A layer, facing, or protective mound of rubble or stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also, the stone used for this purpose.

Ripple. (1) The light fretting or ruffling of a water caused by a breeze. (2) Undulating ridges and furrows, or crests and troughs formed by action of the flow.

Risk. The consequences associated with the probability of flooding attributable to an encroachment. It includes the potential for property loss and hazard to life during the service life of the highway.

Risk Analysis. An economic comparison of design alternatives using expected total costs (construction costs plus risk costs) to determine the alternative with the least expected cost to the public. It must include probable flood-related costs during the service life of the facility for highway operation, maintenance, and repair, for highway aggravated flood damage to other property, and for additional or interrupted highway travel.

Riser. In mountainous terrain where much debris is encountered, the entrance to a culvert sometimes becomes easily clogged. Therefore, a corrugated metal pipe or a structure made of timber or concrete with small perforations, called a riser, is installed vertically to permit entry of water and prohibit the entry of mud and debris. The riser may be increased in height as the need occurs.

River. A large stream, usually active when any streams are flowing in the region.

Rock. (1) Cobble, boulder or quarry stone as a construction material. (2) Hard natural mineral, in formation as in piles of talus.

Rounded Inlet. The edges of a culvert entrance that are rounded for smooth transition which reduces turbulence and increases capacity.

RSP Fabric. (See Filter Fabric).

Rubble. Rough, irregular fragments of rock or concrete.

Runoff. (1) The surface waters that exceed the soil's infiltration rate and depression storage. (2) The portion of precipitation that appears as flow in streams. Drainage or flood discharge which leaves an area as surface flow or a pipeline flow, having reached a channel or pipeline by either surface or subsurface routes.

Runup. The rush of water up a beach or structure, associated with the breaking of a wave. The amount of runup is measured according to the vertical height above still water level that the rush of water reaches.

Sag Culvert (or Sag Pipe). A pipeline with a dip in its grade line crossing over a depression or under a highway, railroad, canal, etc. The term inverted siphon is common but inappropriate as no siphonic action is involved. The term "sag pipe" is suggested as a substitute.

Sand. Granular soil coarser than silt and finer than gravel, ranging in diameter from 0.002 inch to 0.2 inch.

Scour. The result of erosive action of running water, primarily in streams, excavating and carrying away material from the bed and banks. Wearing away by abrasive action.

Scour, General. The removal of material from the bed and banks across all or most of the width of a channel, as a result of a flow contraction which causes increased velocities and bed shear stress.

Scour, Local. Removal of material from the channel bed or banks which is restricted to a minor part of the width of a channel. This scour occurs around piers and embankments and is caused by the actions of vortex systems induced by the obstruction to the flow.

Scour, Natural. Removal of material from the channel bed or banks which occurs in streams with the migration of bed forms, shifting of the thalweg and at bends and natural contractions.

Sea. Ocean or other body of water larger than a lake; state of agitation of any large body of water.

Seawall. A structure separating land and water areas, primarily designed to prevent erosion and other damage due to wave action. (See bulkhead).

Sediment. Fragmentary material that originates from weathering of rocks and is transported by, suspended in, or deposited by water.

Sedimentation. Gravitational deposit of transported material in flowing or standing water.

Seepage. Percolation of underground water thru the banks and into a stream or other body of water.

Seiche. A standing wave oscillation of an enclosed waterbody that continues, pendulum fashion, after the cessation of the originating force, which may have been either seismic or atmospheric.

Seismic Wave. A gravity wave caused by an earthquake.

Sheet Flow. Any flow spread out and not confined; i.e., flow across a flat open field.

Sheet Pile. A pile with a generally slender, flat cross-section that is driven into ground or bottom of a water body and meshed or interlocked with like members to form a wall or bulkhead.

Shoal. A shallow region in flowing or standing water, especially if made shallow by deposition.

Shoaling. Deposition of alluvial material resulting in areas with relatively shallow depth.

Shore. The narrow strip of land in immediate contact with the water, including the zone between high and low water lines. See backshore, foreshore, onshore, offshore, longshore, and nearshore.

Significant Encroachment. A highway encroachment and any direct support of likely base floodplain development that would involve one or more of the following construction or flood related impacts:

- A significant potential for interruption or termination of a transportation facility which is needed for emergency vehicles or provides a community's only evacuation route.
- A significant risk, or
- A significant adverse impact on natural and beneficial floodplain values.

Silt. (1) *Water-Borne Sediment.* Detritus carried in suspension or deposited by flowing water, ranging in diameter from 0.0002 inch to 0.002 inch. The term is generally confined to fine earth, sand, or mud, but is sometimes both suspended and bedload. (2) *Deposits of Water-Borne Material.* As in a reservoir, on a delta, or on floodplains.

Sinuosity. The ratio of the length of the river thalweg to the length of the valley proper.

Skew. When a drainage structure is not normal (perpendicular) to the longitudinal axis of the highway, it is said to be on a skew. The skew angle is the smallest angle between the perpendicular and the axis of the structure.

Slide. Gravitational movement of an unstable mass of earth from its natural position.

Slipout. Gravitational movement of an unstable mass of earth from its constructed position. Applied to embankments and other man-made earthworks.

Slope. (1) Gradient of a stream. (2) Inclination of the face of an embankment, expressed as the ratio of horizontal to vertical projection; or (3) The face of an inclined embankment or cut slope. In hydraulics it is expressed as percent or in decimal form.

Slough. (1) Pronounced SLU. A side or overflow channel in which water is continually present. It is stagnant or slack; also a waterway in a tidal marsh. (2) Pronounced SLUFF. Slide or slipout of a thin mantle of earth, especially in a series of small movements.

Slugflow. Flow in culvert or drainage structure which alternates between full and partly full. Pulsating flow -- mixed water and air.

Soffit. The bottom of the top -- (1) With reference to a bridge, the low point on the underside of the suspended portion of the structure. (2) In a culvert, the uppermost point on the inside of the structure.

Specific Energy. The energy contained in a stream of water, expressed in terms of head, referred to the bed of a stream. It is equal to the mean depth of water plus the velocity head of the mean velocity.

Spur Dike. A structure or embankment projecting a short distance into a stream from the bank and at an angle to deflect flowing water away from critical areas.

Stage. The elevation of a water surface above its minimum; also above or below an established "low water" plane; hence above or below any datum of reference; gage height.

Standing Wave. The motion of swiftly flowing stream water, that resembles a wave, but is formed by decelerating or diverging flow that does not quite produce a hydraulic jump. A term which when used to describe the upper flow regime in alluvial channels, means a vertical oscillation of the water surface between fixed nodes without appreciable progression in either an upstream or downstream direction. To maintain the fixed position, the wave must have a celerity (velocity) equal to the approach velocity in the channel, but in the opposite direction.

Steady Flow. A flow in which the flow rate or quantity of fluid passing a given point per unit of time remains constant.

Stone. Rock or rock-like material; a particle of such material, in any size from pebble to the largest quarried blocks.

Storage. Detention, or retention of water for future flow, naturally in channel and marginal soils or artificially in reservoirs.

Storage Basin. Space for detention or retention of water for future flow, naturally in channel and marginal soils, or artificially in reservoirs.

Storm. A disturbance of the ordinary, average conditions of the atmosphere which, unless specifically qualified, may include any or all meteorological disturbances, such as wind, rain, snow, hail, or thunder.

Storm Drain. That portion of a drainage system expressly for collecting and conveying former surface water in an enclosed conduit. Often referred to as a "storm sewer", storm drains include inlet structures, conduit, junctions, manholes, outfalls and other appurtenances.

Storm Water Management. The recognition of adverse drainage resulting from altered runoff and the solutions resulting from the cooperative efforts of public agencies and the private sector to mitigate, abate, or reverse those adverse results.

Strand. (1) To lodge on bars, banks, or overflow plain, as for drift. (2) Bar of sediment connecting two regions of higher ground.

Stream. Water flowing in a channel or conduit, ranging in size from small creeks to large rivers.

Stream Power. An expression used in predicting bed forms and hence bed load transport in alluvial channels. It is the product of the mean velocity, the specific weight of the water-sediment mixture, the normal depth of flow and the slope.

Stream Response. Changes in the dynamic equilibrium of a stream by any one, or combination of various causes.

Stream Waters. Former surface waters which have entered and now flow in a well defined natural watercourse, together with other waters reaching the stream by direct precipitation or rising from springs in bed or banks of the watercourse. They continue as stream waters as long as they flow in the watercourse, including overflow and multiple channels as well as the ordinary or low-water channel.

Strutting. Elongation of the vertical axis of pipe prior to installing in a trench. After the backfill has been placed around the pipe and compacted, the wires or rods holding the pipe in its distorted shape are removed. Greater side support from the earth is developed when the pipe tends to return to its original shape. Generally used on pipes which because of size or thinness of the metal would tend to deform during construction operations. Arches are strutted diagonally per standard or special plan.

Subcritical Flow. In this state, gravity forces are dominant, so that the flow has a low velocity and is often described as tranquil and streaming. Also, that flow which has a Froude number less than one.

Subdrain. A conduit for collecting and disposing of underground water. It generally consists of a pipe, with perforations in the bottom through which water can enter.

Subsidence. General lowering of land surface by consolidation or removal of underlying soil.

Sump. In drainage, any low area which does not permit the escape of water by gravity flow.

Supercritical Flow. In this state, inertia forces are dominant, so that flow has a high velocity and is usually described as rapid, shooting and torrential. Also, that flow which has a Froude number greater than one.

Support Base Floodplain Development. To encourage, allow, serve, or otherwise facilitate additional base floodplain development. Direct support results from an encroachment, while indirect support results from an action out of the base floodplain.

Surf. The breaking of waves and swell on the foreshore and offshore shoals.

Surface Runoff. The movement of water on earth's surface, whether flow is over surface of ground or in channels.

Surface Waters. Surface waters are those which have been precipitated on the land from the sky or forced to the surface in springs, and which have then spread over the surface of the ground without being collected into a definite body or channel. They appear as puddles, sheet or overland flow, and rills, and continue to be surface waters until they disappear from the surface by infiltration or evaporation, or until by overland or vagrant flow they reach well-defined watercourses or standing bodies of water like lakes or seas.

Surge. A sudden swelling of discharge in unsteady flow.

Suspended Load. Sediment that is supported by the upward components of turbulent currents in a stream and that stay in suspension for appreciable amount of time.

Swale. A shallow, gentle depression in the earth's surface. This tends to collect the waters to some extent and is considered in a sense as a drainage course, although waters in a swale are not considered stream waters.

Swamp. An area of shallow pondage or saturated surface, the water being fresh or acidic and the area usually covered with rank vegetation.

Swell. Waves generated by a distant storm, usually regular and fully harmonic.

Talus. Loose rocks and debris disintegrated from a steep hill or cliff standing at repose along the toe.

Tapered Inlet. A transition to direct the flow of water into a channel or culvert. A smooth transition to increase hydraulic efficiency of an inlet structure.

Terrace. Berm or bench-like earth embankment, with a nearly level plain bounded by rising and falling slopes.

Tetrahedron. Bank protection element, basically composed of 6 steel or concrete struts joined like the edges of a triangular pyramid, together with subdividing struts and tie wires or cables.

Tetrapod. Bank protection element, precast of concrete, consisting of 4 legs joined at a central block, each leg making an angle of 109.5 degrees with the other three, like rays from the center of a tetrahedron to the center of each face.

Texture. Arrangement and interconnection of surface and near-surface particles of terrain or channel perimeter.

Thalweg. The line following the lowest part of a valley, whether under water or not. Usually the line following the deepest part of the bed or channel of a river.

Thread. The central element of a current, continuous along a stream.

Tide. The periodic rising and falling of the ocean and connecting bodies of water that results from gravitational attraction of the moon and sun acting on the rotating earth.

Time of Concentration. The time required for storm runoff to flow from the most remote point, in flow time, of a drainage area to the point under consideration. It is usually associated with the design storm.

Topping. The top layer on horizontal revetments or rock structures; also capping or cap stones.

Training. Control of direction of currents.

Transition. A relatively short reach or conduit leading from one waterway section to another of different width, shape, or slope.

Transport. To carry solid material in a stream in solution, suspension, saltation, or entrainment.

Trash Rack. A grid or screen across a stream designed to catch floating debris.

Trough. Space between wave crests and the water surface below it.

Trunk (or Trunk Line). In a roadway drainage system, the main conduit for transporting the storm waters. This main line is generally quite deep in the ground so that laterals coming from fairly long distances can drain by gravity into the trunk line.

Tsunami. A gravity wave caused by an underwater seismic disturbance (such as sudden faulting, landsliding or volcanic activity).

Turbulence. A state of flow wherein the water is agitated by cross-currents and eddies, as opposed to a condition of flow that is quiet and laminar.

Turbulent Flow. That type of flow in which any particle may move in any direction with respect to any other particle, and in which the head loss is approximately proportional to the square of the velocity.

Undercut. Erosion of the low part of a steep bank so as to compromise stability of the upper part.

Underflow. The downstream flow of water through the permeable deposits that underlie a stream. (1) Movement of water through a pervious subsurface stratum, the flow of percolating water; or water under ice, or under a structure. (2) The rate of flow or discharge of subsurface water.

Undertow. Current outward from a wave-swept shore carrying solid particles swept or scoured from the beach or foreshore.

Unsteady Flow. A flow in which the velocity changes with respect to space and time.

Updrift. The direction opposite that of the predominant movement of littoral materials.

Uplift. Upward hydrostatic pressure on base of an impervious structure.

Velocity. The rate of motion of objects or particles, or of a stream of particles.

Velocity Head. A term used in hydraulics to represent the kinetic energy of flowing water. This "head" is represented by a column of standing water equivalent in potential energy to the kinetic energy of the moving water calculated as ($V^2/2g$) where the "V" represents the velocity in feet per second and "g" represents the potential acceleration due to gravity, in feet per second per second.

Vernal Pools. Seasonally flooded landscape depressions that support distinctive (and many times rare) plant and animal species adapted to periodic or continuous inundation during the wet season, and the absence of either ponded water or wet soil during the dry season.

Wash. Floodplain or active channel of an ephemeral stream, usually in recent alluvium.

Watercourse. A definite channel with bed and banks within which water flows, either continuously or in season. A watercourse is continuous in the direction of flow and may extend laterally beyond the definite banks to include overflow channels contiguous to the ordinary channel. The term does not include artificial channels such as canals and drains, except natural channels trained or restrained by the works of man. Neither does it include depressions or swales through which surface or errant waters pass.

Watershed. The area that contributes surface water runoff into a tributary system or water course.

Water Table. The surface of the groundwater below which the void spaces are completely saturated.

Waterway. (1) That portion of a watercourse which is actually occupied by water (2) A navigable inland body of water.

Wave. (1) An oscillatory movement of water on or near the surface of standing water in which a succession of crests and troughs advance while particles of water follow cyclic paths without advancing. (2) Motion of water in a flowing stream so as to develop the surficial appearance of a wave.

Wave Height. The vertical distance between a wave crest and the preceding trough.

Wave Length. The horizontal distance between similar points on two successive waves (e.g., crest to crest or trough to trough), measured in the direction of wave travel.

Wave Period. The time in which a wave crest travels a distance equal to one wave length. Can be measured as the time for two successive wave crests to pass a fixed point.

Weephole. A hole in a wall, invert, apron, lining, or other solid structure to relieve the pressure of groundwater.

Weir. A low overflow dam or sill for measuring, diverting, or checking flow.

Well. (1) Artificial excavation for withdrawal of water from underground storage. (2) Upward component of velocity in a stream.

Wetland. Those areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas.

Windbreak. Barrier fence or trees to break or deflect the velocity of wind.

Windwave. A wave generated and propelled by wind blowing along the water surface.

Young. Immature, said of a stream on a steep gradient actively scouring its bed toward a more stable grade.

Topic 807 - Selected Drainage References

807.1 Introduction

Hydraulic and drainage related reference publications listed are grouped as to source.

807.2 Federal Highway Administration Hydraulic Publications

Copies of publications identified with an NTIS or GPO number may be ordered as follows:

NTIS - Send a check to:

National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650

GPO - Send a check to:

Superintendent of Documents
Government Printing Office
Washington, D.C. 20402
(202) 783-3238

(1) *Hydraulic Engineering Circulars (HEC).*

HEC No.	Title	Date	FHWA # NTIS #
9	Debris-Control Structures	2005	IF-04-016
14	Hydraulic Design of Energy Dissipators for Culverts and Channels	2006	NHI-06-086
15	Design of Roadside Channels with Flexible Linings	2005	IF-05-114
17	The Design of Encroachments on Flood Plains Using Risk Analysis	1981	EPD-86-112 PB86-182110/AS
18	Evaluating Scour at Bridges	2012	HIF-12-003
20	Stream Stability at Highway Structures	2012	HIF-12-004
21	Bridge Deck Drainage Systems	1993	SA-92-010 PB94-109584
22	Urban Drainage Design Manual	2009	NHI-10-009
23	Bridge Scour and Stream Instability Countermeasures	2009	NHI-09-111 NHI-09-012
24	Highway Stormwater Pump Station Design	2001	NHI-01-007
25	Highways in the Coastal Environment	2008	NHI-07-096
26	Culvert Designer Aquatic Organism Passage	2010	HIF-11-008

(2) *Hydraulic Design Series (HDS).*

HDS No.	Title	Date	FHWA # NTIS #
2	Highway Hydrology	2002	NHI-02-001
3	Design Charts for Open-Channel Flow	1961	EPD-86-102 PB86-179249/AS
4	Introduction to Highway Hydraulics	2008	NHI-08-090
5	Hydraulic Design of Highway Culverts (GPO 050-001-00298-1)	2012	HIF-12-026
6	River Engineering for Highway Encroachments	2001	NHI-01-004
7	Hydraulic Design for Safe Bridges	2012	HIF-12-018

(3) *Implementation Publications.*

Title	Date	FHWA # NTIS #
Structural Design Manual for Improved Inlets and Culverts	1983	IP-83-6 PB84-153485
Culvert Inspection Manual	1986	IP-86-2 PB87-151809

807.3 American Association of State Highway and Transportation Officials (AASHTO)

(1) *Highway Drainage Guidelines*

The Drainage Guidelines is a collection of the guides previously published as individual volumes. These are:

I - Hydraulic Considerations in Highway Planning and Location
II - Hydrology
III - Erosion and Sediment Control in Highway Construction
IV - Hydraulic Design of Culverts
V - The Legal Aspects of Highway Drainage
VI - Hydraulic Analysis and Design of Open Channels
VII - Hydraulic Analysis for the Location and Design of Bridges
VIII- Hydraulic Aspects in Restoration and Upgrading of Highways
IX - Storm Drain Systems
X - Evaluating Highway Effects on Surface Water Environments
XI - Highways along Coastal Zones and Lakeshores
XII - Stormwater Management
XIII - Hydraulics Engineer Training and Career Development
XIV - Culvert Inspection and Rehabilitation
XV - Guidelines for Selecting and Utilizing Hydraulics Engineering Consultants

The current edition may be purchased through AASHTO, 444 North Capitol St., N.W., Suite 225, Washington D.C. 20001.

(2) AASHTO Model Drainage Manual

The Model Drainage Manual (MDM) is a comprehensive document covering a wide variety of transportation related hydraulic design issues. Developed for use by Federal, State, and local agencies, the MDM is a practice oriented document that allows the user agency to adopt the recommended values shown in the manual, or insert their own specific design policies and procedures.

807.4 California Department of Transportation

The following publications are available from the Caltrans Publications Unit, 1900 Royal Oaks Dr., Sacramento, CA 95815. Information on ordering and price can be checked by calling (916) 445-3520.

- Bridge Design Practice Manual
- Manual of Test - Volumes 1, 2, and 3
- Standard Plans
- Standard Specifications

807.5 U.S. Department of Interior - Geological Survey (USGS)

- Magnitude and Frequency of Floods in California - Water Resources Investigation 77-21.
- Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States - Water-Supply Paper 2433.
- Guide For Determining Flood Flow Frequency - Bulletin #17B.
- Water Resources Data for California, Part 1, Volumes 1 and 2.
- Rock Riprap Design for Protection of Stream Channels Near Highway Structures (1987) Volumes 1 and 2 (1987).
- Regional Skew for California, and Flood Frequency for Selected Sites in the Sacramento-San Joaquin River Basin, Based on Data through Water Year 2006 - Scientific Investigations Report 2010-5260.

807.6 U.S. Department of Agriculture - Natural Resources Conservation Service (NRCS)

- Engineering Design Standards.
- Urban Hydrology for Small Watersheds - Technical Release 55

807.7 California Department of Water Resources

The California Department of Water Resources provides intensity, duration, and frequency data from the California Department of Water Resources network of rain gauges at the following website: http://www.water.ca.gov/floodmgmt/hafoo/hb/csm/engineering/

807.8 University of California - Institute of Transportation and Traffic Engineering (ITTE)

- Street and Highway Drainage - Course Notes, Volumes 1 and 2.

807.9 U.S. Army Corps of Engineers

Publications and computer programs, too numerous to list, are available from the Water Resources Support Center. A publication catalog may be obtained by contacting the Hydrologic Engineering Center of the Corp, 609 Second St., Davis, CA 95616. The U. S. Army Corps of Engineers publications website address is: http://www.usace.army.mil/inet/usace-docs/.

Topic 808 – Selected Computer Programs

Table 808.1 below presents a software vs. capabilities matrix for hydrologic/hydraulic software packages that have been reviewed and deemed compatible with Departmental procedures. Where Caltrans drainage facilities connect or impact facilities that are owned by others, the affected Local Agency may require the Department to use a specific program that is not listed below. When the use of other computer programs is requested, a comparison with the results using the appropriate program from Table 808.1 should be made. However, when work is performed on projects under Caltrans' jurisdiction, either internally, or by others, if a program not listed in Table 808.1 is used, it should be demonstrated that the computations are based on the same principles that are used in the programs listed in Table 808.1. For information on Local Agency hydraulic computer program requirements, the District Hydraulics Branch should be contacted. It is the responsibility of the user to ensure that the version of the program being used from Table 808.1 is current.

Table 808.1

Summary of Related Computer Programs and Web Applications

	Storm Drains	Hydrology	Water Surface Profiles	Culverts	Roadside /Median Channels	Pavement Drainage	Pond Routing
FHWA Hydraulic Toolbox					x	x	
TR-55		x					
HEC-HMS (2)		x					x
HY-8				x			
HEC-RAS (1)			x				
FESWMS			x				
WMS		x		x			x
NOAA Atlas 14		x					
USGS StreamStats		x					
AutoDesk Civil 3D/Hydraflow	x	x				x	x

NOTES:

(1) The data that was used by FEMA to establish water surface elevations (usually HEC-2) must be used to develop a duplicate effective model for FEMA floodplain analysis. For more information contact FEMA or the Local Agency.

(2) HEC-1 has been superseded by HEC-HMS by the U.S. Army Corps of Engineers.

Special circumstances may dictate the use of alternative methods/programs. Any such use should be performed under direction and with approval of the District Hydraulics Engineer.

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.

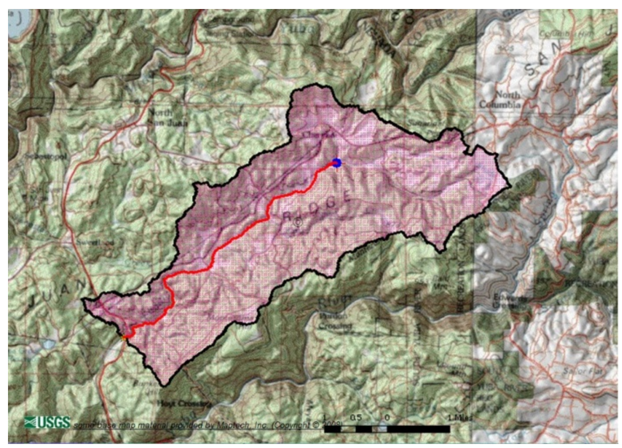
Topic 812 - Basin Characteristics

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

- 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

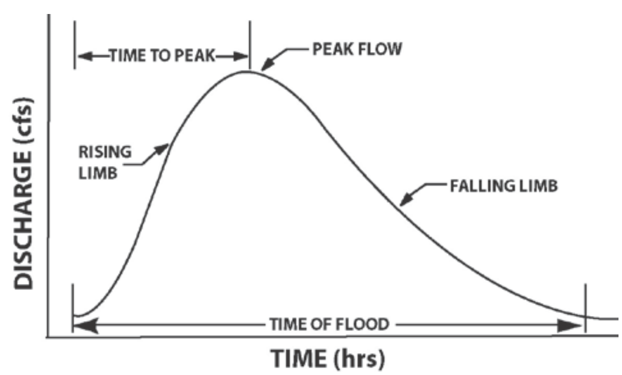
Water which accumulates and ponds in low points or depressions in the soil surface with no possibility for escape as runoff is in retention storage. Where water is moving over the land it is in detention storage. Detained water, as opposed to retained water, contributes to runoff.

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

See Index 819.4 for a general discussion of hydrograph methods.

816.6 Time of Concentration (Tc) and Travel Time (Tt)

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 feet.

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 P_2 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	*n*
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
Grass	
Short grass prairie	0.15
Dense grass	0.24
Bermuda Grass	0.41
Woods(1)	
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:

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

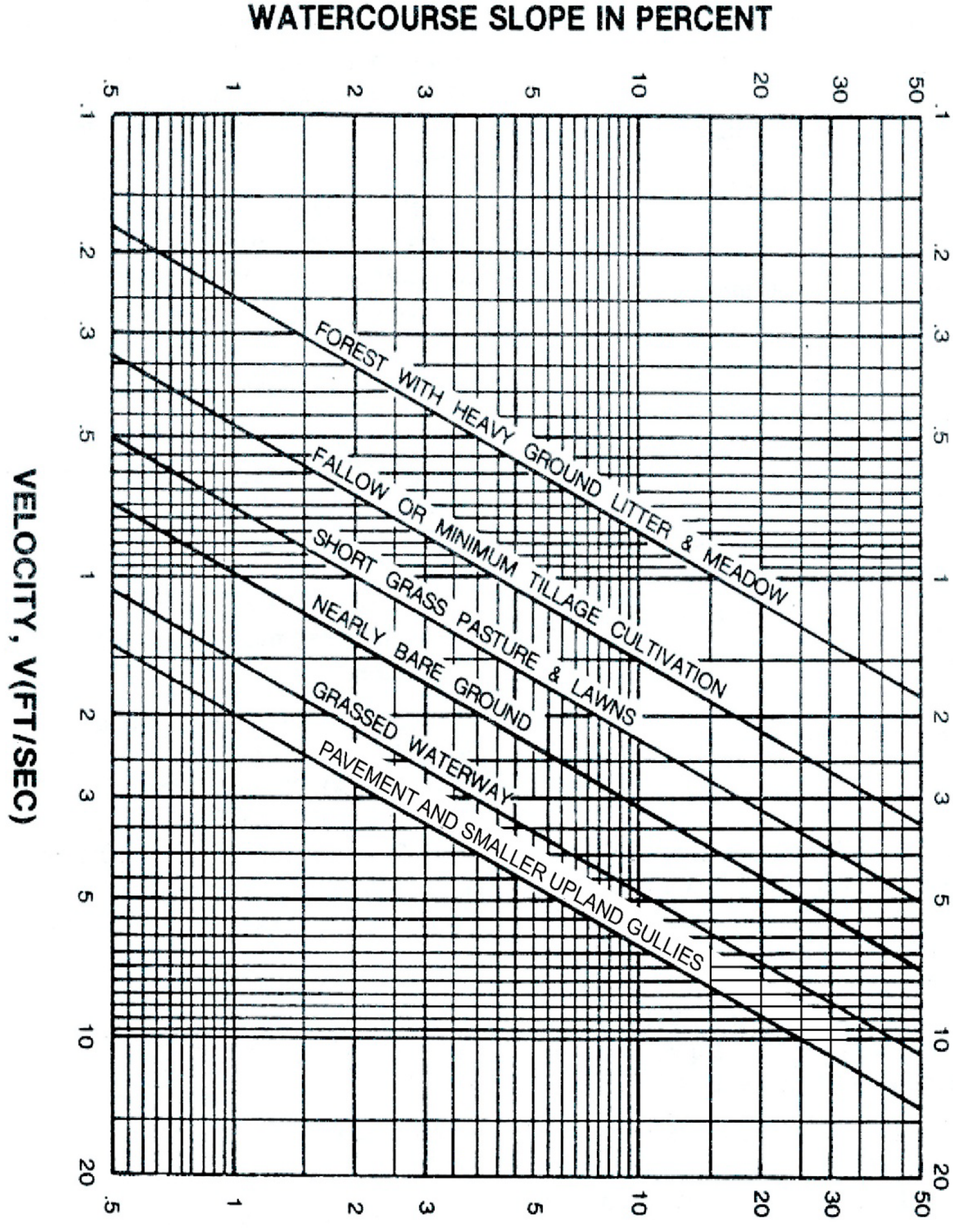
Where S is the slope in percent and k is an intercept coefficient depending on land cover as shown in Table 816.6B. It is assumed that the depth range is 0.1 to 0.2 feet, except for grassed waterways, where the depth range is 0.1 to 0.4 feet,

Table 816.6B Intercept Coefficients for Shallow Concentrated Flow

Land cover/Flow regime	k
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



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.

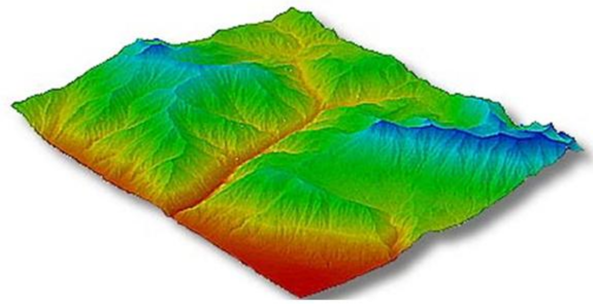
Appropriate values for "n", the coefficient of roughness in the Manning’s equation, may be found in most hydrology or hydraulics texts and reference books. Table 866.3A gives some "n" values for lined and unlined channels, gutters, and medians. Procedures for selecting an appropriate hydraulic roughness 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:

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

- 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*

(a) 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.

(b) 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.

(c) At a gaged site, weighted estimates of peak discharges based on the station flood-frequency relation and the regional

Figure 819.2A

Runoff Coefficients for Undeveloped Areas

Watershed Types

	Extreme	High	Normal	Low
Relief	.28 -.35 Steep, rugged terrain with average slopes above 30%	.20 -.28 Hilly, with average slopes of 10 to 30%	.14 -.20 Rolling, with average slopes of 5 to 10%	.08 -.14 Relatively flat land, with average slopes of 0 to 5%
Soil Infiltration	.12 -.16 No effective soil cover, either rock or thin soil mantle of negligible infiltration capacity	.08 -.12 Slow to take up water, clay or shallow loam soils of low infiltration capacity, imperfectly or poorly drained	.06 -.08 Normal; well drained light or medium textured soils, sandy loams, silt and silt loams	.04 -.06 High; deep sand or other soil that takes up water readily, very light well drained soils
Vegetal Cover	.12 -.16 No effective plant cover, bare or very sparse cover	.08 -.12 Poor to fair; clean cultivation crops, or poor natural cover, less than 20% of drainage area over good cover	.06 -.08 Fair to good; about 50% of area in good grassland or woodland, not more than 50% of area in cultivated crops	.04 -.06 Good to excellent; about 90% of drainage area in good grassland, woodland or equivalent cover
Surface Storage	.10 -.12 Negligible surface depression few and shallow; drainageways steep and small, no marshes	.08 -.10 Low; well defined system of small drainageways; no ponds or marshes	.06 -.08 Normal; considerable surface depression storage; lakes and pond marshes	.04 -.06 High; surface storage, high; drainage system not sharply defined; large floodplain storage or large number of ponds or marshes

Given	An undeveloped watershed consisting of; 1) rolling terrain with average slopes of 5%, 2) clay type soils, 3) good grassland area, and 4) normal surface depressions.	Solution: Relief 0.14 Soil Infiltration 0.08 Vegetal Cover 0.04 Surface Storage <u>0.06</u> C = 0.32
Find	The runoff coefficient, C, for the above watershed.	

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.

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.

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)
$Q_2 = 0.00459A^{0.856}P^{2.58}$	$Q_2 = 3.60A^{0.672}P^{0.753}$
$Q_5 = 0.0984A^{0.852}P^{1.97}$	$Q_5 = 7.43A^{0.739}P^{0.872}$
$Q_{10} = 0.460A^{0.846}P^{1.66}$	$Q_{10} = 6.56A^{0.783}P^{1.07}$
$Q_{25} = 2.13A^{0.842}P^{1.34}$	$Q_{25} = 4.71A^{0.832}P^{1.32}$
$Q_{50} = 5.32A^{0.840}P^{1.15}$	$Q_{50} = 3.84A^{0.864}P^{1.47}$
$Q_{100} = 11.0A^{0.840}P^{0.994}$	$Q_{100} = 3.28A^{0.891}P^{1.59}$

Q = Peak discharge in CFS, subscript indicates recurrence interval, in years

A = Drainage area, in square miles

P = Mean annual precipitation, in inches (Use link to Table 2) http://pubs.usgs.gov/sir/2012/5113/

E = Mean basin elevation, in feet

Region	Drainage Area (A), mi^2	Mean Annual Precipitation (P), in.	Mean Basin Elevation (E), ft.
North Coast	0.04 – 3200	20 – 125	-
Lahontan (1)	0.45 – 1500	13 – 85	-
Sierra Nevada	0.07 – 2000	15 – 100	90 – 11,000
Central Coast	0.11 – 4600	7 – 46	-
South Coast	0.04 – 850	10 – 45	-
Desert (2)	N/A	N/A	-

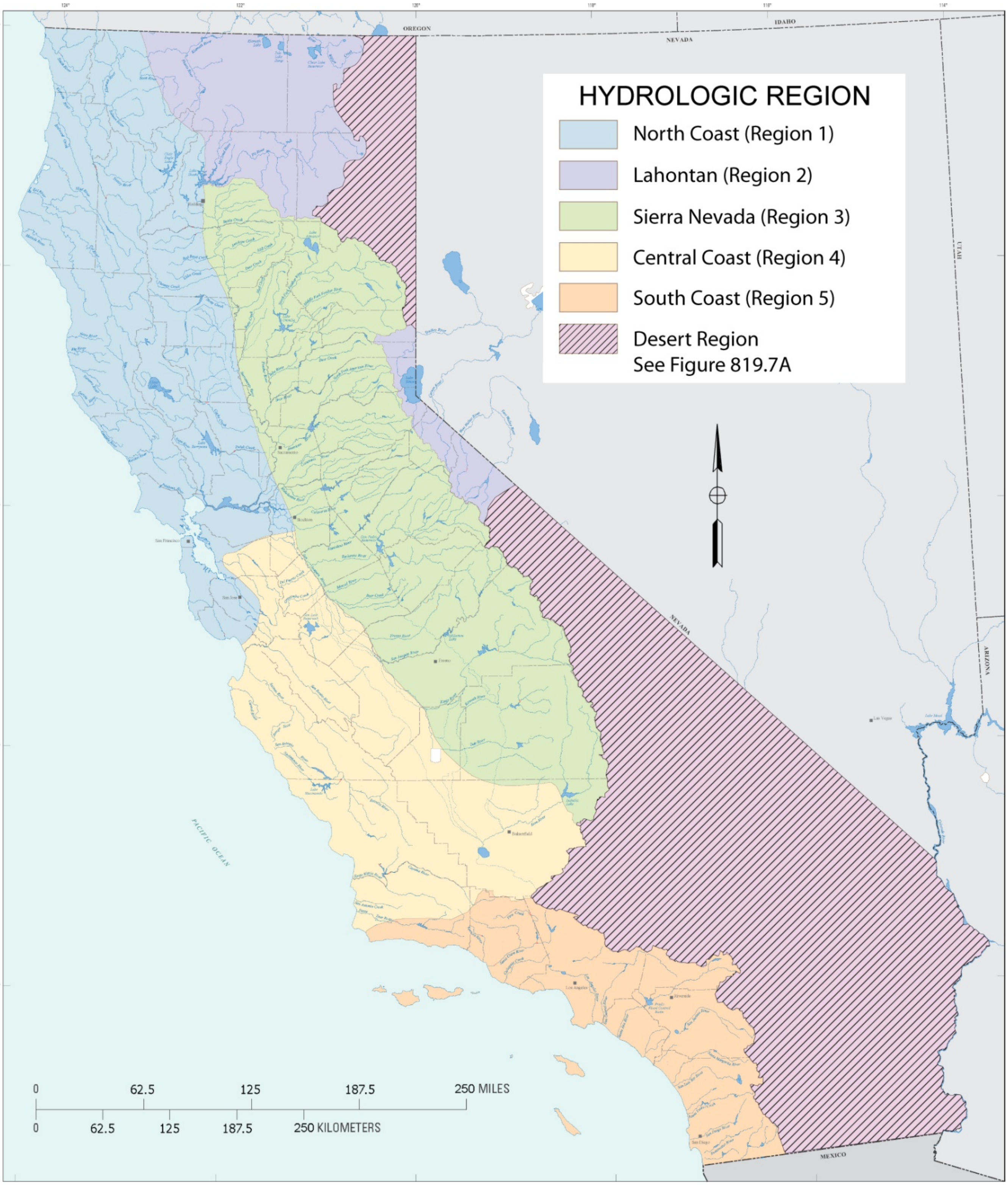
NOTES:

(1) See Index 819.7 for hydrologic procedures for those portions of the Northeast Region classified as desert.

(2) USGS equations not recommended. See Index 819.7.

Figure 819.2C

Regional Flood-Frequency Regions



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.

    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

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

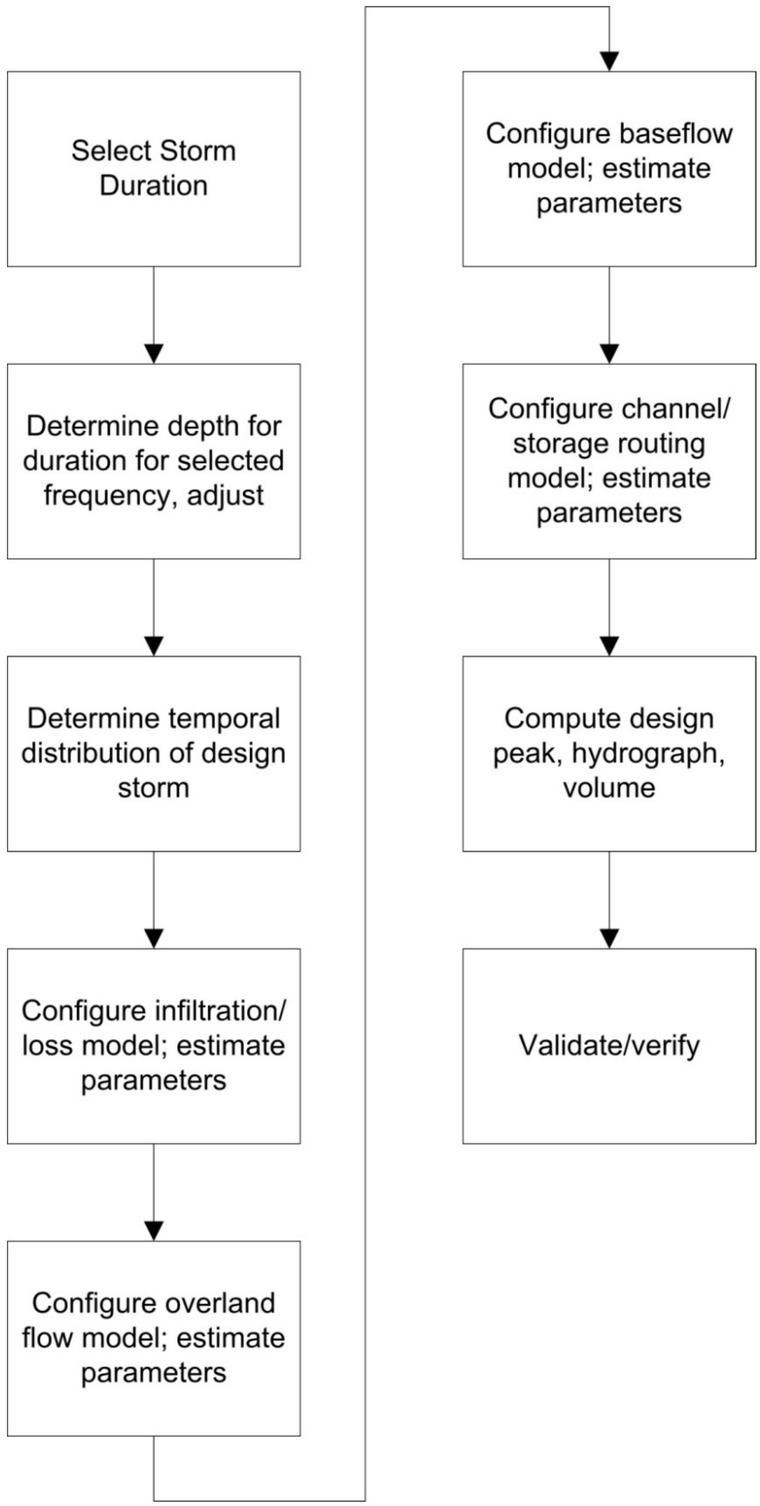
For more information see; Chapters 4, 5, 6, 7 and 8 of the user guide for HEC-HMS. See: http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_(CPD-74B).pdf

819.5 Transfer of Data

Often the highway engineer is confronted with the problem where stream flow and rainfall data are not

Figure 819.4A

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



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.

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 mi^2), and intense rainfall, which may result

Table 819.5A
Summary of Methods for Estimating Design Discharge

METHOD	ASSUMPTIONS	DATA NEEDS
Rational	• Small catchment (< 320 acres) • Concentration time < 1 hour • Storm duration >or = 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	• 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)	• 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	• 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	• 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	• Similar hydrologic characteristics • Channel storage	Discharge and area for gaged watershed Area for ungaged watershed

* Magnitude and Frequency of Floods in California

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/mi^2 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 mi^2).

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 $\leq$ 20 mi^2 – Drainage areas less than 20 mi^2 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 mi^2 & ≤ 100 mi^2 – For drainage areas between 20 mi^2 and 100 mi^2, 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 mi^2 – Since general storms usually cover a larger area and have a longer duration, for drainage areas greater than 100 mi^2, 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-miunte 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

(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

Figure 819.7A

Desert Regions in California

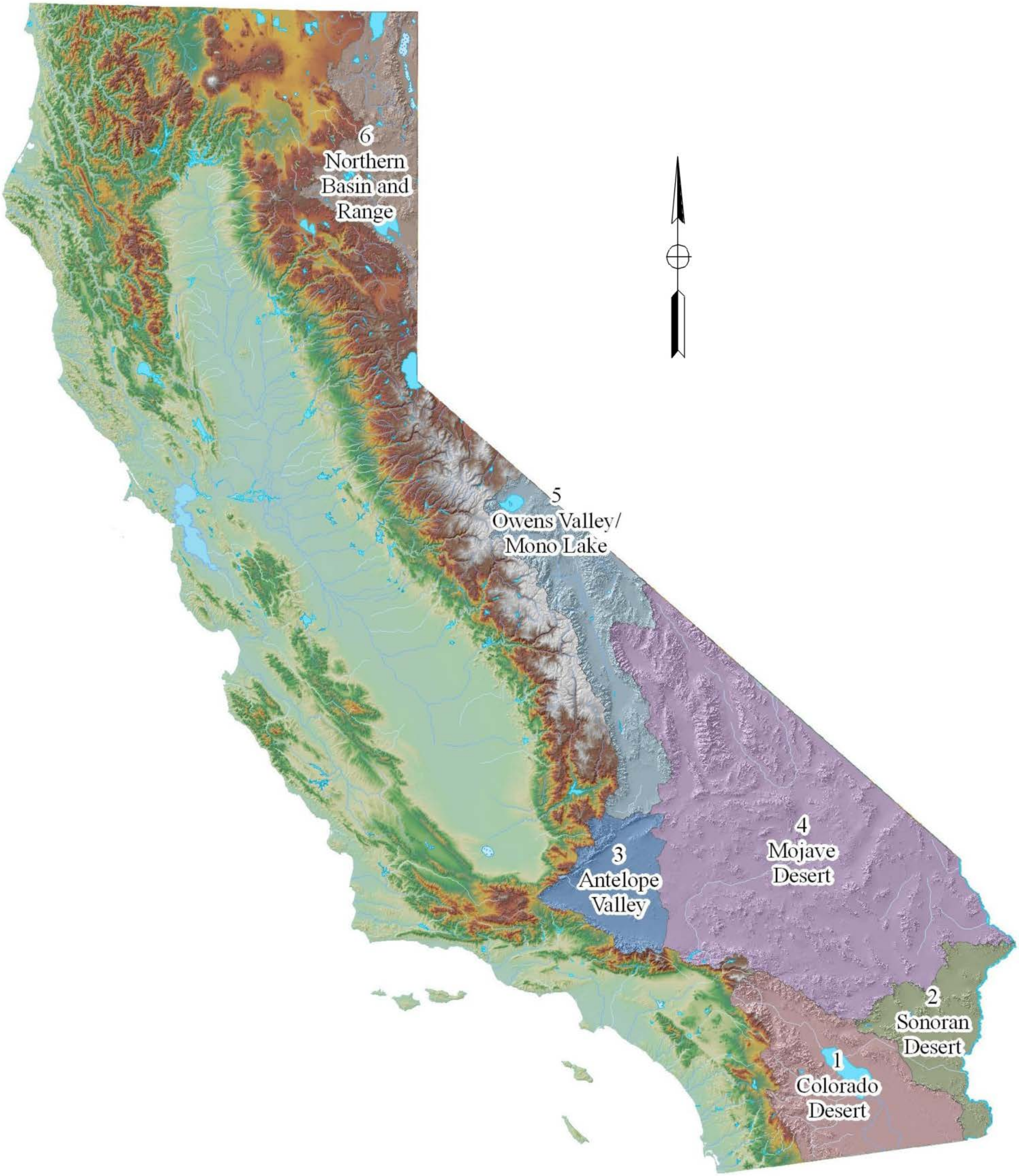


Table 819.7A

Regional Regression Equations for California's Desert Regions

Region(s)	Associated Regression Equations	
Colorado Desert Sonoran Desert Antelope Valley Mojave Desert	$Q_2 = 8.57A^{0.5668}$ $Q_5 = 80.32A^{0.541}$ $Q_{10} = 146.33A^{0.549}$	$Q_{25} = 291.04A^{0.5939}$ $Q_{50} = 397.82A^{0.6189}$ $Q_{100} = 557.31A^{0.6619}$
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 ($\leq$ 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

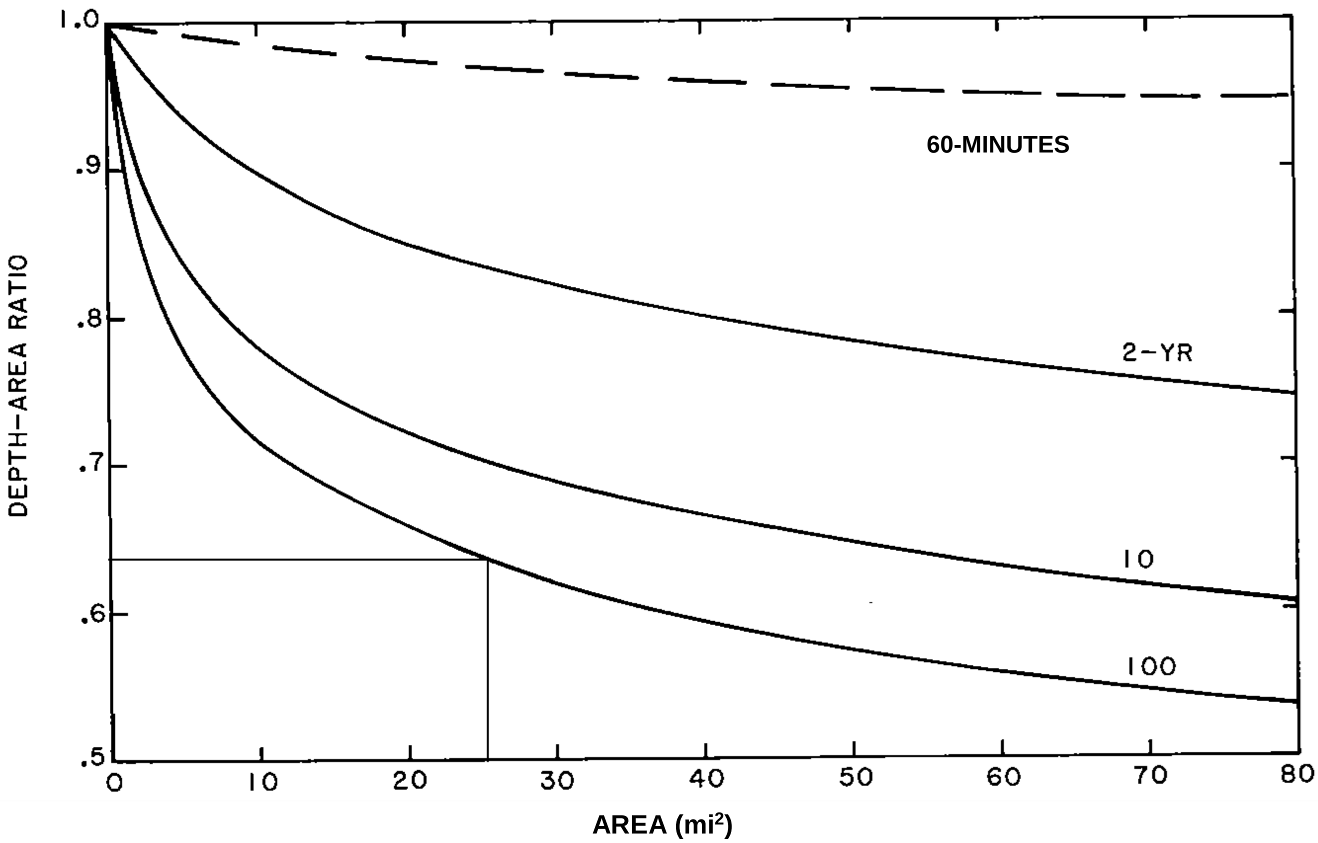
HYDRO-40 (http://www.nws.noaa.gov/oh/hdsc/PF_related_studies/TechnicalMemorandum_HYDRO40.pdf); and (2) the depth curves in NOAA Atlas 2. The general consensus is that the depth curves from HDRO-40 better represent the desert areas of California, and are recommended for the southern desert regions (Colorado 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 mi^2, 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).

Figure 819.7B

Example Depth-Area Reduction Curve



(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:

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.

$$CN_{AMCI} = \frac{4.2CN_{AMCII}}{10 - 0.058CN_{AMCII}}$$

$$CN_{AMCIII} = \frac{23CN_{AMCII}}{10 + 0.13CN_{AMCII}}$$

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.

(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 Hydrogoraph

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 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 ungaged 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

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

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 ungaged drainage areas for which the hydrologic characteristics can be determined, and a unit hydrograph applicable to the ungaged 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.

(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

Figure 819.7C

San Bernardino County Hydrograph for Desert Areas

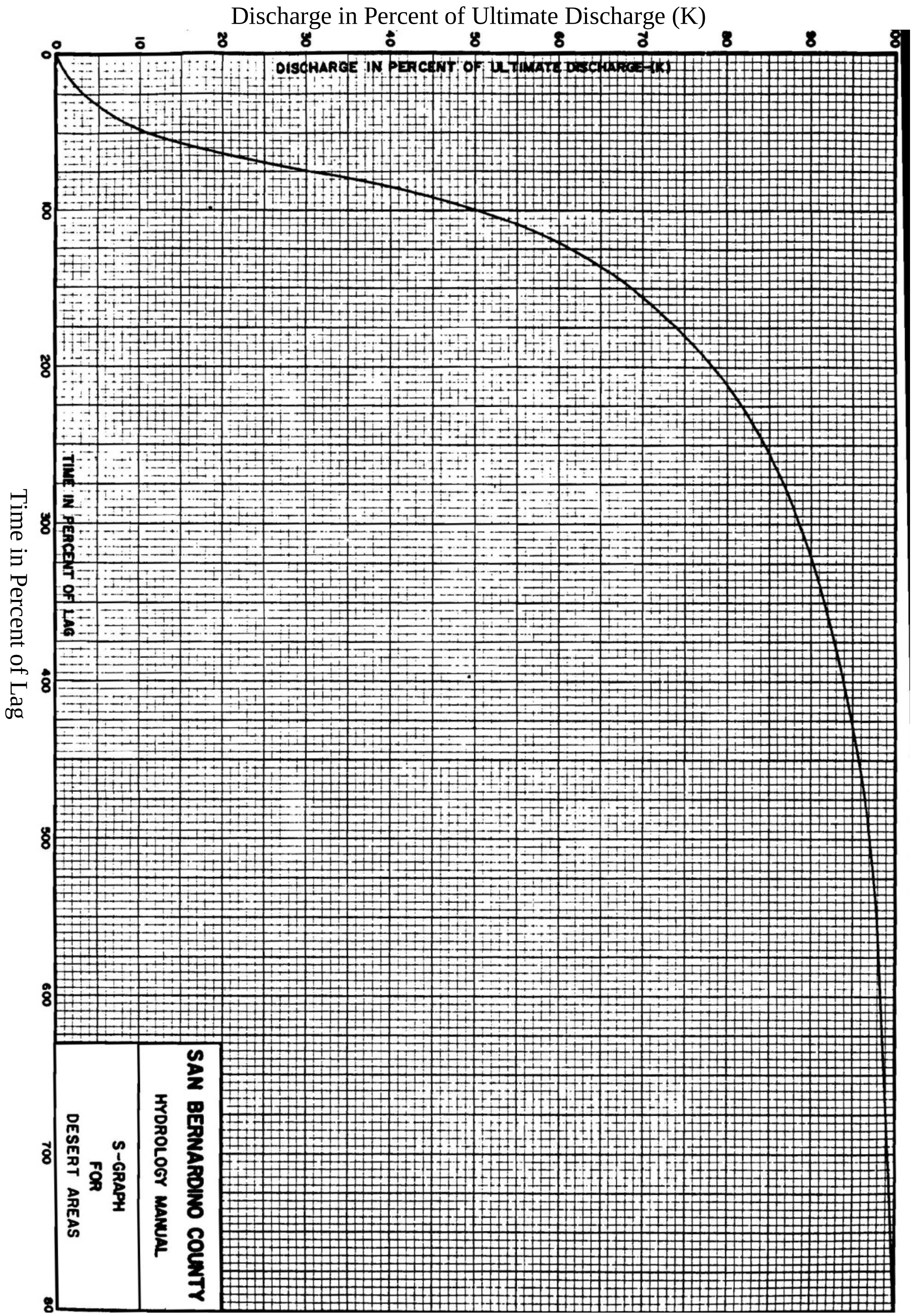


Figure 819.7D

USBR Example S-Graph

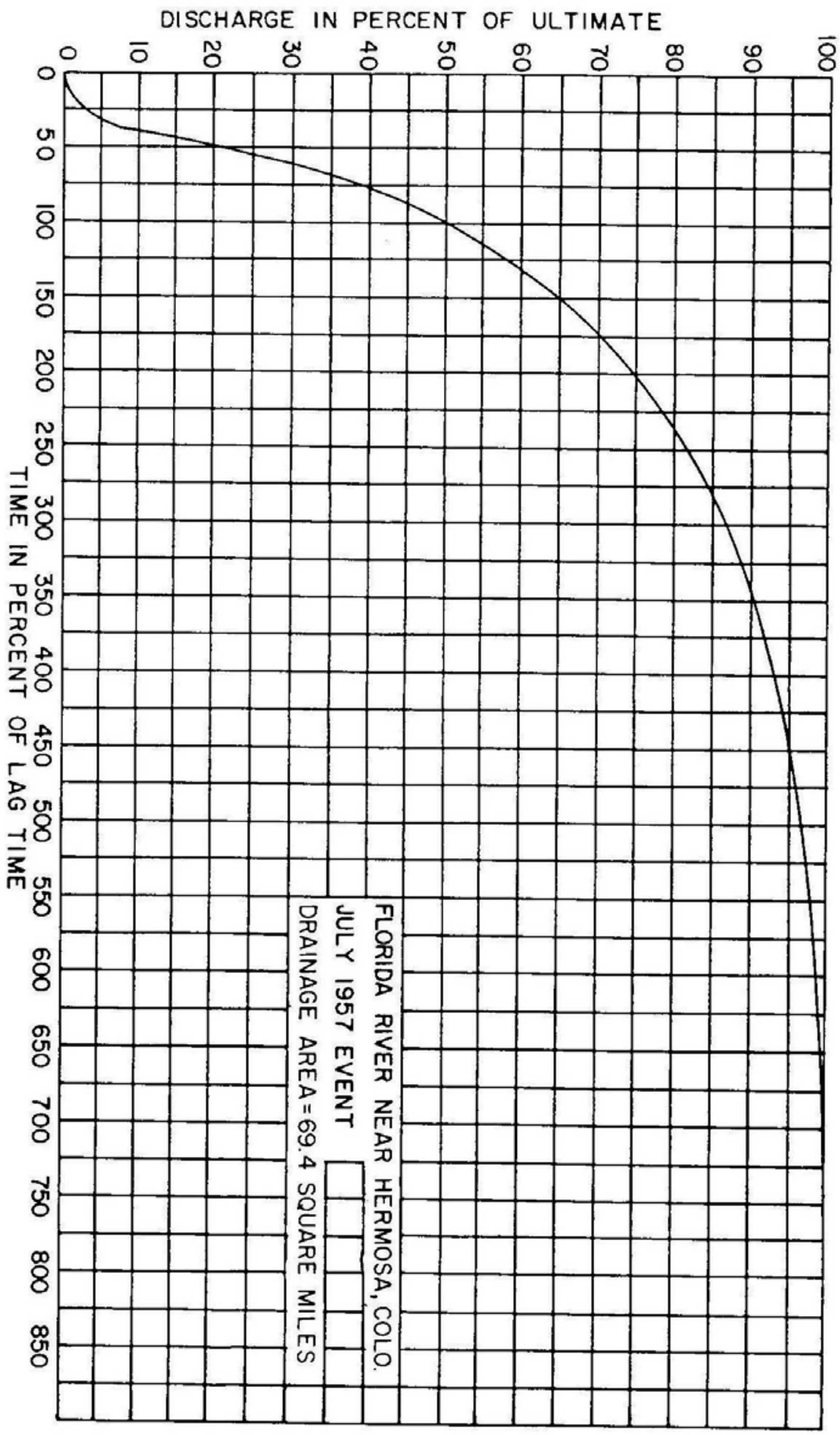


Table 819.7F

Channel Routing Methods

Routing Method	Pros	Cons
Kinematic Wave	▪ 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.	▪ Cannot handle hydrograph attenuation, significant overbank storage, and backwater effects.
Modified Puls	▪ Known as storage routing or level-pool routing. ▪ Can handle backwater effects through the storage-discharge relationship.	▪ Need to use hydraulic model to define the required storage-outflow relationship.
Muskingum	▪ Directly accommodates the looped relationship between storage and outflow. ▪ A linear routing technique that uses coefficients to account for hydrograph timing and diffusion.	▪ 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	▪ 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.	▪ It cannot account for backwater effects. ▪ Not very applicable for routing a very rapidly rising hydrograph through a flat channel.

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

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.

(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.

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 mi^2 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.

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.

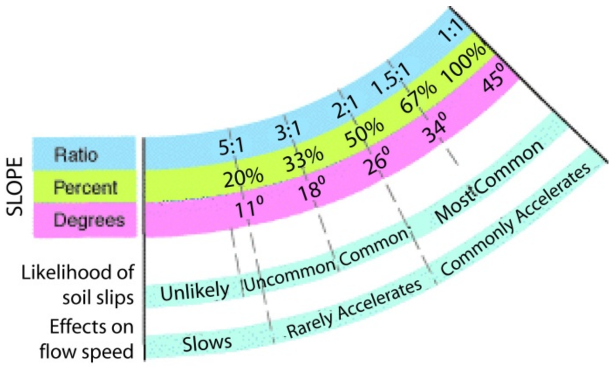
Table 819.7I

Bulking Factors & Types of Sediment Flow

Sediment Flow Type	Bulking Factor	Sediment Concentration by Weight	Sediment Concentration by Volume
		(100% by WT = 1 x 10^6 ppm)	(specific gravity = 2.65)
	0	0	0
Normal Streamflow	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

Figure 819.7E

Soil Slips vs. Slope Angle



(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.

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).

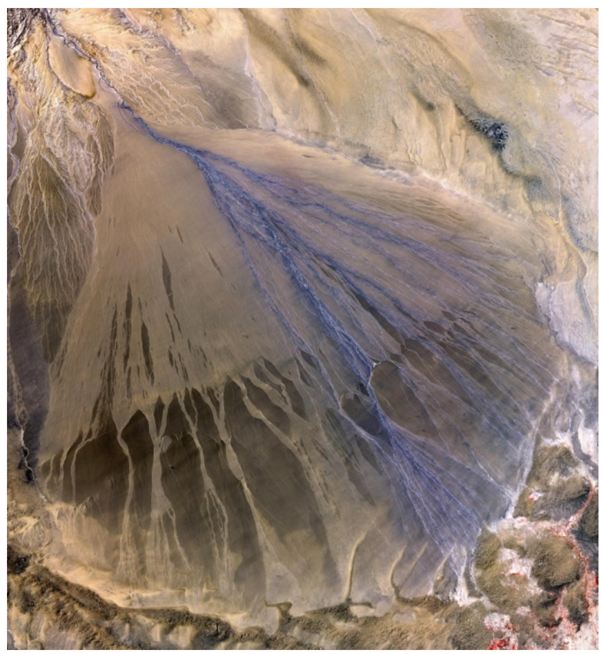
(d) Alluvial Fans

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 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 mi^2 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 mi^2, 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 calculated 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

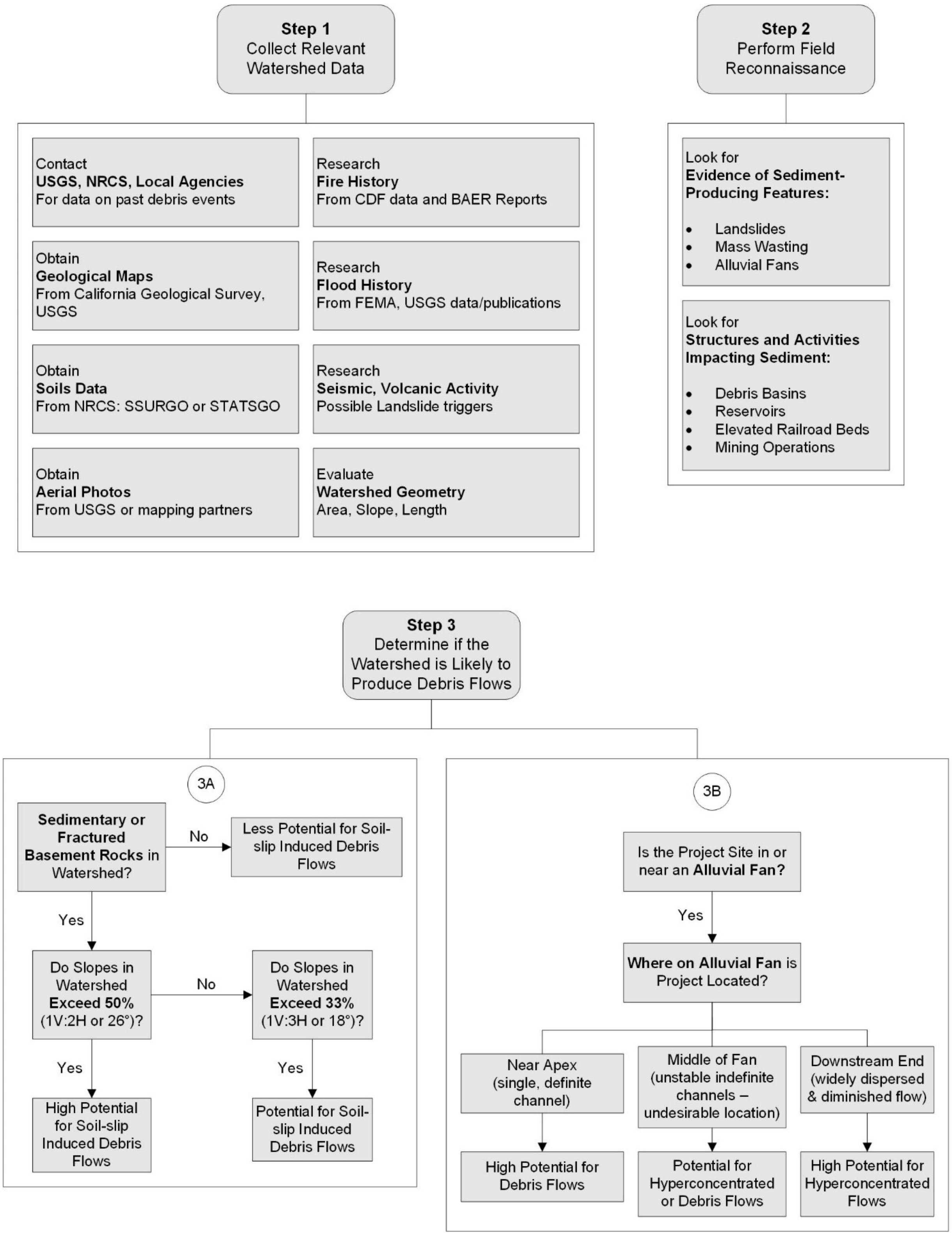
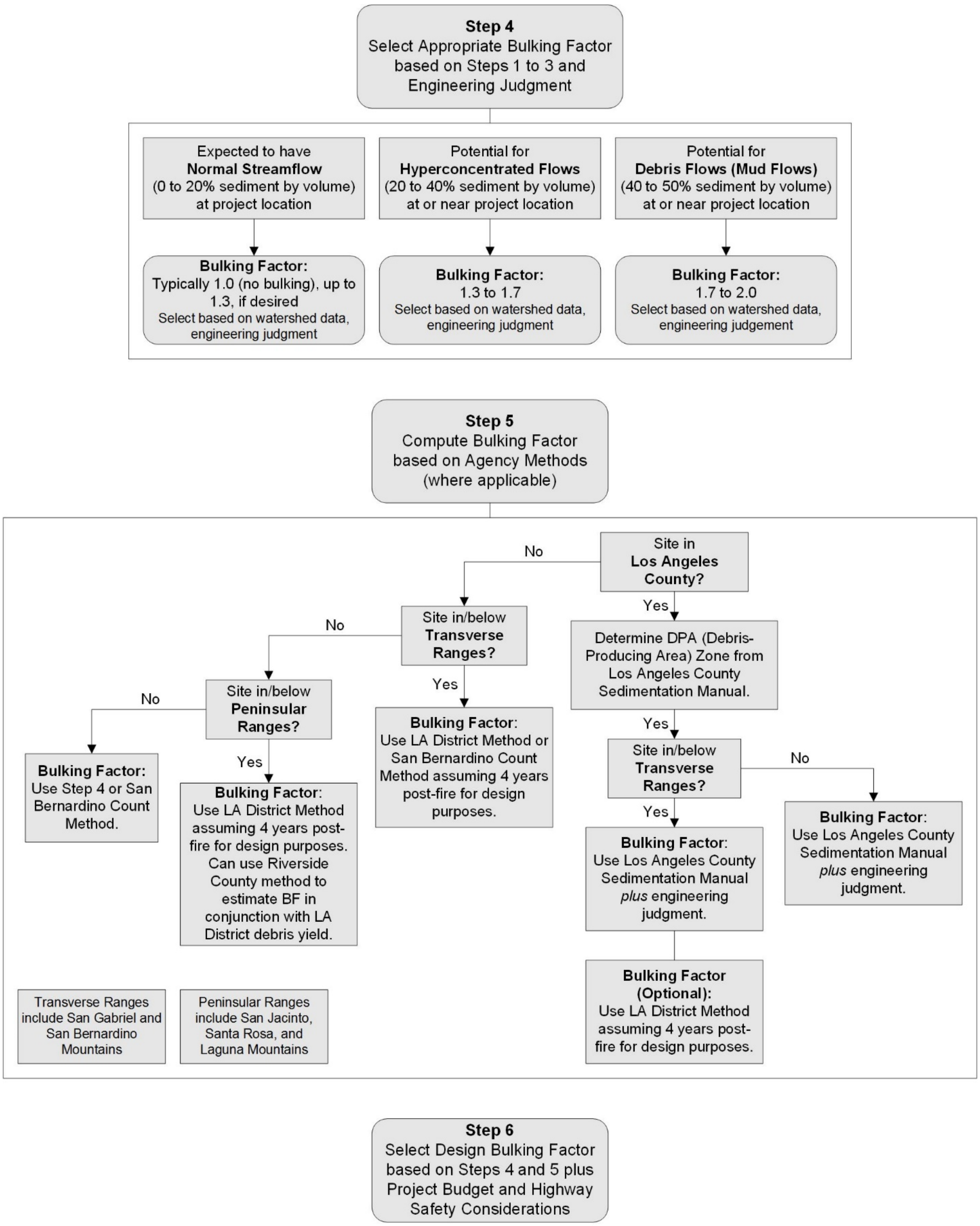


Figure 819.7H

Recommended Bulking Factor Selection Process (Cont'd)



CHAPTER 820
CROSS DRAINAGE

Topic 821 - General

Index 821.1 - Introduction

Cross drainage involves the conveyance of surface water and stream flow across or from the highway right of way. This is accomplished by providing either a culvert or a bridge to convey the flow from one side of the roadway to the other side or past some other type of flow obstruction.

In addition to the hydraulic function, a culvert must carry construction and highway traffic and earth loads. Culvert design, therefore, involves both hydraulic and structural design. This section of the manual is basically concerned with the hydraulic design of culverts. Both the hydraulic and structural designs must be consistent with good engineering practice and economics. An itemized listing of good drainage design objectives and economic factors to be considered are listed in Index 801.4. Information on strength requirements, height of fill tables, and other physical characteristics of alternate culvert shapes and materials may be found in Chapter 850, Physical Standards.

More complete information on hydraulic principles and engineering techniques of culvert design may be found in the FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts". Key aspects of culvert design and a good overview of the subject are more fully discussed in the AASHTO Highway Drainage Guidelines.

Structures measuring more than 20 feet along the roadway centerline are conventionally classified as bridges, assigned a bridge number, and maintained and inspected by the Division of Structures. However, some structures classified as bridges are designed hydraulically and structurally as culverts. Some examples are certain multi-barreled box culverts and arch culverts. Culverts, as distinguished from bridges, are usually covered with embankment and have structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert.

Bridges are not designed to take advantage of submergence to increase hydraulic capacity even though some are designed to be inundated under flood conditions. For economic and hydraulic efficiency, culverts should be designed to operate with the inlets submerged during flood flows, if conditions permit. At many locations, either a bridge or a culvert will fulfill both the structural and hydraulic requirements of the stream crossing. Structure choice at these locations should be based on construction and maintenance costs, risk of failure, risk of property damage, traffic safety, and environmental and aesthetic considerations.

Culverts are usually considered minor structures, but they are of great importance to adequate drainage and the integrity of the highway facility. Although the cost of individual culverts is relatively small, the cumulative cost of culvert construction constitutes a substantial share of the total cost of highway construction. Similarly, the cost of maintaining highway drainage features is substantial, and culvert maintenance is a large share of these costs. Improved service to the public and a reduction in the total cost of highway construction and maintenance can be achieved by judicious choice of design criteria and careful attention to the hydraulic design of each culvert.

821.2 Hydrologic Considerations

Before the hydraulic design of a culvert or bridge can begin, the design discharge, the quantity (Q) of water in cubic feet per second, that the facility may reasonably be expected to convey must be estimated. The most important step is to establish the appropriate design storm or flood frequency for the specific site and prevailing conditions. Refer to Chapter 810, Hydrology and specifically Topics 818 and 819 for useful information on hydrological analysis methods and considerations.

When empirical methods are used to estimate the peak rate of runoff, design Q, for important culverts, it is recommended that at least two methods be tried. By comparing results a more reliable discharge estimate for the drainage basin may be obtained. This is more important for large basins having areas in excess of 320 acres than for small basins.

821.3 Selection of Design Flood

As discussed in Index 818.2, there are two recognized alternatives to selecting the design flood frequency (probability of exceedance) in the hydraulic design of bridges and culverts. They are:

- By policy - using a preselected recurrence interval.
- By analysis - using the recurrence interval that is most cost effective and best satisfies the specific site conditions and associated risks.

Although either of these alternatives may be used exclusive of the other, in actual practice both alternatives are often considered and used jointly to select the flood frequency for hydraulic design. For culverts and bridges, apply the following general rules for first consideration in the process for ultimate selection of the design flood.

(1) *Bridges.* The basic rule for the hydraulic design of bridges (but not including those culvert structures that meet the definition of a bridge) is that they should pass a 2 percent probability flood (50-year). Freeboard, vertical clearance between the lowest structural member and the water surface elevation of the design flood, sufficient to accommodate the effects of bedload and debris should be provided. Alternatively, a waterway area sufficient to pass the 1 percent probability flood without freeboard should be provided. Two feet of freeboard is often assumed for preliminary bridge designs. The effects of bedload and debris should be considered in the design of the bridge waterway.

(2) *Culverts.* There are two primary design frequencies that should be considered:

- A 10% probability flood (10-year) without causing the headwater elevation to rise above the inlet top of the culvert and,
- A 1% probability flood (100-year) with-out headwaters rising above an elevation that would cause objectionable backwater depths or outlet velocities.

The designer must use discretion in applying the above criteria. Design floods selected on this basis may not be the most appropriate for specific project site locations or conditions. The cost of providing facilities to pass peak discharges suggested by these criteria need to be balanced against potential damage to the highway and adjacent properties upstream and downstream of the site. The selection of a design flood with a lesser or greater peak discharge may be warranted and justified by economic analysis. A more frequent design flood than a 4% probability of exceedance (25-year) should not be used for the hydraulic design of culverts under freeways and other highways of major importance. Alternatively, where predictive data is limited, or where the risks associated with drainage facility failure are high, the greatest flood of record or other suitably large event should be evaluated by the designer.

When channels or drainage facilities under the jurisdiction of local flood control agencies or Corps of Engineers are involved, the design flood must be determined through negotiations with the agencies involved.

821.4 Headwater and Tailwater

(1) *Headwater.* The term, headwater, refers to the depth of the upstream water surface measured from the invert of the culvert entrance. Any culvert which constricts the natural stream flow will cause a rise in the upstream water surface.

It is not always economical or practical to utilize all the available head. This applies particularly to situations where debris must pass through the culvert, where a headwater pool cannot be tolerated, or where the natural gradient is steep and high outlet velocities are objectionable.

The available head may be limited by the fill height, damage to the highway facility, or the effects of ponding on upstream property. The extent of ponding should be brought to the attention of all interested functions, including Project Development, Maintenance, and Right of Way.

Full use of available head may develop some vortex related problems and also develop 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 disjointing 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 decreases 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 disjointing 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 "downdrain" 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.

CHAPTER 830 TRANSPORTATION FACILITY DRAINAGE

Topic 831 - General

Index 831.1 - Basic Concepts

Roadway drainage involves the collection, conveyance, removal, and disposal of surface water runoff from the traveled way, shoulders, sidewalks, and adjoining areas defined in Index 62.1(7) as comprising the roadway. Roadway drainage is also concerned with the handling of water from the following additional sources:

- Surface water from outside the right of way and not confined to channels that would reach the traveled way if not intercepted.
- Crossroads or streets.
- Irrigation of landscaped areas.

The design of roadway drainage systems often involves consideration of the problems associated with inadequate drainage of the adjacent or surrounding area. Cooperative drainage improvement projects with the responsible local agency may offer the best overall solution. Cooperative agreements are more fully discussed under Index 803.2

Some of the major considerations of good roadway drainage design are:

- Facility user safety.
- Convenience to vehicular, bicycle and pedestrian traffic.
- Aesthetics.
- Flooding of the transportation facility and adjacent property.
- Subgrade infiltration.
- Potential erosion, pollution and other environmental concerns.
- Economy of construction.
- Economy of maintenance.

This section involves the hydraulic design fundamentals necessary for properly sizing and locating standard highway drainage features such as:

- Asphalt dikes and gutters.
- Concrete curbs and gutters.
- Median drains.
- Roadside ditches
- Overside drains.
- Drop inlets.
- Storm drains.

Removal of storm water from highway pavement surfaces and median areas is more fully discussed in FHWA Hydraulic Engineering Circular No. 22, "Urban Drainage Design Manual". HEC 22 includes discussion of the effects of roadway geometry on pavement drainage; the philosophy of design storm frequency and design spread selection; storm runoff estimating methods; pavement and bridge deck inlets; and flow in gutters. Charts and procedures are provided for the hydraulic analysis and design of roadway drainage features.

831.2 Highway Grade Line

In flat terrain, roadway drainage considerations often control the longitudinal grade line of the highway. A grade line that assures the desirable goal of keeping the traveled way free of flooding can usually be established for new freeway projects and rural conventional highways.

For multilane urban highways with nearly continuous dike or curb along the shoulder or parking area, it is seldom practical to design the highway with a gutter section which will contain all of the runoff even from frequent rains. For this reason the gutter and shoulder combination, and often partial or full width of the traveled way, are used to convey the runoff to inlets.

831.3 Design Storm and Water Spread

Before the hydraulic adequacy of roadway drainage facilities can be analyzed, the quantity of water (design Q) that the facility may reasonably be expected to convey must be estimated. The most important, and often the most difficult phase of this task is the selection of an appropriate design storm frequency for the specific project, location or site under consideration. In order for a design frequency to be meaningful criteria for roadway drainage design, it must be tied to an acceptable tolerance of flooding. Design water spread, encroachment upon the roadbed or adjacent property, is the tolerance of flooding directly related to roadway drainage design. Allowing too little spread is uneconomical in design and too much spread may result in unsafe driving conditions.

To optimize economy in roadway drainage, the allowable water spread should vary, depending on the type of project being designed. Because of the effect of splash and spray on motorist visibility and vehicle control, high volume roads with high speed traffic cannot tolerate as much water spread as urban streets. Likewise, the allowable water spread should be minimized on urban streets where a large number of pedestrians use adjacent sidewalks and pedestrian crosswalks. Consideration should be given to the element of motorist surprise when encountering intermittent puddles rather than a continuous encroachment of water on the driving lane. Eccentric forces are exerted on a vehicle when one side encounters water in the lane and the other side does not.

The probability of exceedance of the design storm and the acceptable tolerance to flooding depends on the importance of the highway and risks involved. Selection of the design storm and water spread parameters on rehabilitation and reconstruction are generally controlled by existing constraints.

In addition to the major roadway drainage considerations previously listed, the following more specific factors are to be considered in establishing the project design storm:

- Highway type
- Traffic volume
- Design speed
- Local standards

The following geometric and design features of the highway directly affect establishment of the project design water spread:

- Cross slope
- Longitudinal slope
- Number of lanes
- Width of shoulders
- Height of curb and dike
- Parking lanes
- Bus/Transit pullouts and loading areas

Desirable limits for water spread with respect to design storm probability of exceedance are given in Table 831.3. The parameters shown are considered minimum roadway drainage design standards for new freeway construction and for all State highways with depressed sections which require pumping. Local conditions may justify less stringent criteria than the table parameters for conventional highways. Exceptions should be documented by memo to the project file.

It is often advantageous, to both the State and the local agency, for highway drainage and street drainage to be compatible. This is particularly true in urban areas and rapidly developing suburban areas where a conventional highway is, or will become, part of the street network. Street drainage criteria adopted by a local agency are generally based on the hydrologic events peculiar to a geographical area. Local drainage standards that satisfy the needs of the community, usually provide reasonable traffic safety and flood risk considerations commensurate with those normally expected for conventional highways in urban areas.

831.4 Other Considerations

(1) *Sheet Flow.* Concentrations of sheet flow across roadways are to be avoided. As a general rule, no more than 0.10 cubic feet per second should be allowed to concentrate and flow across a roadway. Particular attention should be given to reversal points of superelevation where shoulder and gutter slopes may direct flows across the roadway and gore areas.

(2) *Stage Construction.* All permanent features of roadway drainage systems should be designed and constructed for the ultimate highway facility.

Table 831.3

Desirable Roadway Drainage Guidelines

	DESIGN STORM		DESIGN WATER SPREAD		
HIGHWAY Type/Category/Feature	4% (25 yrs)	10% (10 yrs)	Shldr or Parking Lane	1/2 Outer Lane	Local Standard
FREEWAYS					
Through traffic lanes, branch connections, and other major ramp connections.	X	--	X	--	--
Minor ramps.	--	X	X	--	--
Frontage roads.	--	X	--	--	X
CONVENTIONAL HIGHWAYS					
High volume, multilane Speeds over 45 mph.	X	--	X	--	--
High volume, multilane Speeds 45 mph and under.	--	X	--	X	--
Low volume, rural Speeds over 45 mph.	X	--	X	--	--
Urban Speeds 45 mph and under.	--	X	--	--	X

ALL STATE HIGHWAYS

Depressed Sections That Require Pumping:

Use a 2% (50 yrs) design storm for freeways and conventionalState highways. Design water spread at depressed sections should not exceed that of adjacent roadway sections. A 4% (25 yr) design storm may be used on local streets or road undercrossings that require pumping.

(3) *Landscaping.* Runoff from existing or proposed landscaping, including excess irrigation water runoff, must be considered.

(4) *Groundwater.* Groundwater is subsurface water within a permeable strata. Depending upon recharge and withdrawal rates the level of the groundwater table can fluctuate greatly, over a period of a few months or over periods of many years. Consideration should be given to recent history (several years of abnormally wet or dry conditions) as well as the possibility of revised practices by local water districts (either increased pumping or increased recharge).

Pipes located in areas where contact with groundwater within their design life is likely should have watertight joints. If groundwater contact is likely and the surrounding soils are highly erodible (fine grained sand, silty sand and sandy silt/silt of limited cohesion) consideration should be given to wrapping the pipe joint with filter fabric. The fabric should cover a length of 4 feet along the pipe, centered on the joint. Groundwater at or above the drainage system elevation will lead to infiltration. Where this is undesirable, either joint systems capable of resisting the hydrostatic pressure, or dewatering measures, should be incorporated into the design. The design of groundwater control measures must be coordinated with Geotechnical Services in the Division of Engineering Services.

(5) *Hydroplaning.* Hydroplaning is the separation of the tire from the road surface by a thin layer of liquid (usually water) on the pavement. The liquid separates the tire from the pavement because of viscosity (viscous hydroplaning), dynamic lift (dynamic hydroplaning), or a combination of the two. Since water offers little shear resistance, the tire loses its tractive ability and the driver has a loss of control of the vehicle. At locations where there is a potential for hydroplaning, a careful review of the wet weather accident rates should be made using information obtained from the District Traffic Branch. Typical situations that should be evaluated for hydroplaning potential are:

* Where three (3) lanes or more are sloped in the same direction (see Topic 833).
* Where the longitudinal grade and or cross slope are less than minimum (Refer to Index 204.3 for minimum grade and Indexes 301.2 and 302.2 for cross slope).
* Where there are poor pavement conditions (rutting, depressions, inadequate roughness).
* Where water is allowed to concentrate prior to being directed across the travel lanes (see Index 831.4(1)).
* Where re-striping projects will reduce shoulder widths where dike, curb or concrete barrier are present.

These situations may also be present on median widening projects or projects involving pavement rehabilitation and or lane addition on multi-lane highways or freeways.

Speed and tire pressure appear to be a significant factors in the occurrence of hydroplaning, therefore, it is considered to be the driver’s responsibility to exercise prudence and caution when driving during wet conditions (California Basic Speed Law).

Designers do not have control over all of the factors involved in hydroplaning. However, remedial measures may be included in development of a project to reduce hydroplaning potential. The following is provided as guidance for the designer as practical measures to consider:

(1) Pavement Sheet Flow

- Maximize transverse slope (see Topic 833)
- Maximize pavement roughness
- Use of graded course (porous pavements)

(2) Gutter Flow

- Limit water spread to Table 831.3
- Maximize interception of gutter flow above superelevation transitions (see Index 837.3)

(3) Sag Areas

- 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.

(3) *Side Gutters.* These are triangular gutters adjoining the shoulder as shown in Figures 307.2 and 307.5. The main purpose of the 3 feet wide side gutter is to prevent runoff from the cut slopes on the high side of superelevation from flowing across the roadbeds. The use of side gutters in tangent alignment should be avoided where possible. Local drainage conditions, such as in snow areas, may require their use on either tangent or curved alignment in cut sections. In snow areas it may be necessary to increase the width of side gutters from 3 feet to 6 feet. The slope from the edge of the shoulder to the bottom of the gutter should be no steeper than 6:1. The structural section for paved side gutters should be adequate to support maintenance equipment loads.

(4) *Dikes.* Dikes placed adjoining the shoulder, as shown in Figures 307.2, 307.4, and 307.5, provide a paved triangular gutter within the shoulder area. For conditions governing their use, see Index 303.3.

(5) *Chart Solutions.* Charts for solutions to triangular channel flow problems are contained in FHWA Hydraulic Engineering Circular No. 22, "Urban Drainage Design Manual".

834.4 Overside Drains

The purpose of overside drains, sometimes called slope drains, is to protect slopes against erosion. They convey down the slope drainage which is collected from the roadbed, the tops of cuts, or from benches in cut or fill slopes. They may be pipes, flumes or paved spillways.

(1) *Spacing and Location.* The spacing and location of overside drains depend on the configuration of the ground, the highway profile, the quantity of flow and the limitations on flooding stated in Table 831.3. When possible, overside drains should be positioned at the lower end of cut sections. Diversion from one watershed to another should be avoided. If diversion becomes necessary, care should be used in the manner in which this diverted water is disposed.

Overside drains which would be conspicuous or placed in landscaped areas should be concealed by burial or other means.

(2) *Type and Requirement.* Following are details of various types of overside drains and requirements for their use:

(a) Pipe Downdrains. Metal and plastic pipes are adaptable to any slope. They should be used where side slopes are 4:1 or steeper. Long pipe downdrains should be anchored.

The minimum pipe diameter is 8 inches but large flows, debris, or long pipe installations may dictate a larger diameter.

Watertight joints are necessary to prevent leakage which causes slope erosion. Economy in long, high capacity downdrains is achieved by using a pipe taper in the initial reach. Pipe tapers should insure improved flow characteristics and permit use of a smaller diameter pipe below the taper. See Standard Plan D87-A for details.

(b) Flume Downdrains. These are rectangular corrugated metal flumes with a tapered entrance. See the Standard Plan D87-D for details. They are best adapted to slopes that are 2:1 or flatter but if used on 1.5:1 slopes, lengths over 60 feet are not recommended. Abrupt changes in alignment or grade should be avoided. Flume downdrains should be depressed so that the top of the flume is flush with the fill slope.

(c) Paved Spillways. Permanent paved spillways should only be used when the side slopes are flatter than 4:1. On steeper slopes a more positive type of overside drain such as a pipe downdrain should be used.

Temporary paved spillways are effective in preserving raw fill slopes that are 6:1 or flatter in friable soils during the period when protective growth is being established. Paved spillways should be spaced so that a dike 2 inches high placed at the outer edge of the paved shoulder will effectively confine drainage between spillways. When it is necessary to place a spillway on curved alignment, attention must be given to possible overtopping at the bends. See Index 868.2(3) for discussion of superelevation of the water surface.

(3) *Entrance Standards.* Entrance tapers for pipes and flume downdrains are detailed on the Standard Plans. Pipe entrance tapers should be depressed at least 6 inches.

The local depressions called "paved gutter flares" on the Standard Plans are to be used at all entrance tapers. See Standard Plans D87-A and D87-D for details and Index 837.5 for further discussion on local depressions.

In areas where local depressions would decrease safety the use of flush grate inlets or short sections of slotted drain for entrance structures may be necessary.

(4) *Outlet Treatment.* Where excessive erosion at an overside drain outlet is anticipated, a simple energy dissipater should be employed. Preference should be given to inexpensive expedients such as an apron of broken concrete or rock, a short section of pipe placed with its axis vertical with the lowermost 6 inches filled with coarse gravel or rock, or a horizontal tee section which is usually adequate for downdrain discharges.

(5) *Anchorage*. For slopes flatter than 3:1 overside drains do not need to be anchored. For slopes 3:1 or steeper overside drains should be anchored with 6 foot pipe stakes as shown on the Standard Plans to prevent undue strain on the entrance taper or pipe ends. For drains over 150 feet long, and where the slope is steeper than 2:1, cable anchorage should be considered as shown on the Standard Plans. Where the cable would be buried and in contact with soil, a solid galvanized rod should be used the buried portion and a cable, attached to the rod, used for the exposed portion. Beyond the buried portion, a slip joint must be provided when the installation exceeds 60 feet in length. Regard-less of pipe length or steepness of slope, where there is a potential for hillside movement cable anchorage should be considered.

When cable anchorage is used as shown on the Standard Plans, the maximum allowable downdrain lengths shall be 200 feet for a slope of 1.5:1 and 250 feet for a slope of 2:1. For pipe diameters greater than 24 inches, or downdrains to be placed on slopes steeper than 1.5:1, special designs are required. Where there is an abrupt change in direction of flow, such as at the elbow or a tee section downstream of the end of the cable anchorage system, specially designed thrust blocks should be considered.

(6) *Drainage on Benches.* Drainage from benches in cut and fill slopes should be removed at intervals ranging from 300 feet to 500 feet.

(7) *Selection of Types.* Pipe and flume downdrains may consist of either corrugated steel, corrugated aluminum, or any other approved material that meets the minimum design service life required under Chapter 850. Refer to Index 855.2 for additional discussion on limitations of abrasive resistance of aluminum pipe culverts.

Topic 835 - Dikes and Berms

835.1 General

Dikes and berms are to be used only as necessary to confine drainage and protect side slopes susceptible to erosion.

835.2 Earth Berms

(Text Later)

835.3 Dikes

Details of dikes are shown on Standard Plan A87. See Topic 303 for a detailed discussion on the types and placement considerations for dikes.

Topic 836 - Curbs and Gutters

836.1 General

The primary reason for constructing curbs and gutters may be for delineation or pedestrian traffic rather than for drainage considerations. Refer to Topic 303 for further discussion and Standard Plan A87 for details on concrete curbs and gutters.

Whatever the justification for constructing curbs and gutters, they will usually have an effect on surface water runoff and result in becoming a roadway drainage design consideration.

836.2 Gutter Design

(1) *Capacity.* Gutters and drainage facilities are to be designed to keep flooding within the limits given in Table 831.3. Easy solutions to gutter flow problems can be obtained by using the charts contained in FHWA Hydraulic Engineering Circular No. 22, "Urban Drainage Design Manual" which applies to triangular channels and other shapes illustrated in the charts. Parked cars reduce gutter capacity and also can cause water to shoot over the curb. The downstream ends of driveway ramps can also cause water to flow over the curb. As a rule of thumb, gutter capacity should be determined on a depth equal to 0.5 the curb height for grades up to 10 percent and 0.4 the curb height for grades over 10 percent in locations where parking is allowed or where driveways are constructed.

(2) *Grade and Cross Slope.* The longitudinal grade of curbs and gutters is controlled by the highway grade line as discussed under Index 831.2.

The cross slope of standard gutters is typically 8.33 percent toward the curb. Pavement slopes on superelevated roadways extend the full width of the gutter, except that gutter slopes on the low side should be not less than 8.33 percent. Because they cut down gutter capacity and severely reduce inlet efficiency, cross slopes flatter than 8.33 percent should be avoided, except where gutters are adjacent to curb ramps where ADA requirements limit the slope to a maximum of 5 percent.

(3) *Curbed Intersections.* If pedestrian traffic is a ruling factor, intersection drainage presents the following alternatives to be weighed as to effectiveness and economy.

(a) Intercept the whole flow upstream of the crosswalk.

(b) Intercept a part of the water and allow the overflow to cross the intersection. The width of flow should be controlled so that pedestrian traffic is not unduly hampered.

(c) If flow is small, pass the entire flow across the intersecting street in a valley gutter.

(4) *Valley Gutters.* Valley gutters across the traveled way of the highway should not be used. Valley gutters may be used across intersecting streets and driveways, however, at intersections with high traffic volumes on all approaches, it is desirable to intercept all gutter flow upstream of the intersection and avoid the use of valley gutters. Valley gutters are also undesirable along streets where speeds are relatively high. In locations of frequent intermittent low flows, the use of valley gutters with slotted drains should be considered. In general, the total width of gutters should not exceed 6 feet and cross slopes should not exceed 3 percent. Two percent is suggested where more than nominal speeds are involved.

Topic 837 - Inlet Design

837.1 General

The basic features of standard storm drain inlets are shown in Figure 837.1. Full details appear on Standard Plan D72 through D75, D98-A and D98-B. The variety of standard designs available is considered sufficient to any drainage situation; hence, the use of nonstandard inlets should be rare.

837.2 Inlet Types

From an operating standpoint, there are five main groups of inlets; these are:

(1) *Curb-Opening.* Curb opening inlets have an opening parallel to the direction of flow in the gutter. This inlet group is adapted to curb and gutter installations. The curb opening is most effective with flows carrying floating debris. As the gutter grade steepens, their interception capacity decreases. Hence, they are commonly used on grades flatter than 3 percent.

When curb opening inlets are used on urban highways other than fenced freeways, a 3/4 inch plain round protection bar is placed horizontally across any curb or wall opening whose height is 7 inches or more. The unsupported length of bar should not exceed 7 feet. Use of the protection bar on streets or roads under other jurisdiction is to be governed by the desires of the responsible authorities.

The Type OS and OL inlets are only used with Type A or B curbs. A checkered steel plate cover is provided for maintenance access.

The Type OS inlet has a curb opening 3.5 feet long. Since a fast flow tends to overshoot such a short opening, it should be used with caution on grades above 3 percent.

The Type OL inlet is a high capacity unit in which the length of curb opening ranges from 7 feet to 21 feet.

(2) *Grate.* Grate inlets provide a grate opening in the gutter or waterway. As a class, grate inlets perform satisfactorily over a wide range of gutter grades. Their main disadvantage is that they are easily clogged by floating trash and should not be used without a curb opening where total interception of flow is required. They merit preference over the curb opening type on grades of 3 percent or more. Gutter depressions, discussed under Index 837.5, increase the capacity of grate inlets. Grate inlets may also be used at locations where a gutter depression is not desirable. See the Standard Plans for grate details.

Locate grate inlets away from areas where bicycles or pedestrians are anticipated whenever possible. Grate designs that are allowed where bicycle and pedestrian traffic occurs have smaller openings and are more easily clogged by trash and debris and are less efficient at intercepting flow. Additional measures may be necessary to mitigate the increased potential for clogging.

The grate types depicted on Standard Plan D77B must be used if bicycle traffic can be expected. Many highways do not prohibit bicycle traffic, but have inlets where bicycle traffic would not be expected to occur (e.g., freeway median). In such instances, the designer may consider use of grates from Standard Plan D77A. The table of final pay weights on Standard Plan D77B indicates the acceptable grate types to be used for each listed type of inlet.

If grate inlets must be placed within a pedestrian path of travel, the grate must be compliant with the Americans with Disabilities Act (ADA) regulations which limit the maximum opening in the direction of pedestrian travel to no more than 0.5 inch. Presently, the only standard grating which meets such restrictive spacing criterion is the slotted corrugated steel pipe with heel guard, as shown in the Standard Plans. Because small openings have an increased potential for clogging, a minimum clogging factor of 50 percent should be assumed; however,that factor should be increased in areas prone to significant debris. Other options which may be considered are grated line drains with specialty grates (see the Standard Plans for grated line drain details, and refer to manufacturers catalogs for special application grates) or specially designed grates for standard inlets. The use of specially designed grates is a nonstandard design that must be approved by the Office of State Highway Drainage Design prior to submittal of PS&E.

(3) *Combination.* Combination inlets provide both a curb opening and a grate. These are high capacity inlets which make use of the advantages offered by both kinds of openings.

(a) Type GO and GDO. These types of inlets have a curb opening directly opposite the grate. The GDO inlet has two grates placed side by side and is designed for intercepting a wide flow. A typical use of these inlets would be in a sag location either in a curb and gutter installation or within a shoulder fringed by a dike.When usedas the surface inlet for a pumping installation, the trash rack shownontheStandardPlanD74Bisprovided.

(b) Type GOL. This is called a sweeper inlet because the curb opening precedes the grate. It is particularly useful as a trash interceptor during the initial phases of a storm. When used in a grade sag, the sweeper inlet can be modified by providing a curb opening on both sides of the grate.

(4) *Pipe.* Pipe drop inlets are made of a commercial pipe section of concrete or corrugated metal. As a class, they develop a high capacity and are generally the most economical type. This type of inlet is intended for uses outside the roadbed at locations that

will not be subjected to normal highway wheel loads.

Two kinds of inlets are provided; a wall opening and a grate top. The wall opening inlet should only be used at protected locations where it is unlikely to be hit by an out of control vehicle.

(a) Wall Opening Intake. This opening is placed normal to the direction of surface flow. It develops a high capacity unaffected by the grade of the approach waterway. The inlet capacity is increased by depressing the opening; also by providing additional openings oriented to intercept flows from different directions. When used as the surface intake to a pumping installation, a trash rack across the opening is required. See Standard Plans for pipe inlet details. Because this type of inlet projects above grade, its use should be avoided in areas subject to traffic leaving the roadway.

(b) Grate Intake. The grate intake intercepts water from any direction. For maximum efficiency, however, the grate bars must be in the direction of greatest surface flow. Being round, it is most effective for flows that are deepest at the center, as in a valley median.

(5) *Slotted Drains.* This type of inlet is made of corrugated metal or polyethylene pipe with a continuous slot on top. This type of inlet can be used in flush, all paved medians with superelevated sections to prevent sheet flow from crossing the centerline of the highway. Short sections of slotted drain may be used as an alternate solution to a grate catch basin in the median or edge of shoulder.

Drop inlets or other type of cleanout should be provided at intervals of about 100 feet.

(6) *Grated Line Drains.*This type of inlet is made of monolithic polymer concrete with a ductile iron frame and grate on top. This type of inlet can be used as an alternative at the locations described under slotted drains, preferably in shoulder areas away from traffic loading. However, additional locations may include localized flat areas of pavement at private and public intersections, superelevation transitions, along shoulders where widening causes a decrease to allowable water spread, tollbooth approaches, ramp termini, parking lots and on the high side of superelevation in snow and ice country to minimize black ice and sheet flow from snow melt. Removable grates should not be placed where subject to traffic.

Short sections of grated line drain may be used in conjunction with an existing drainage inlet as a supplement in sag locations. However, based on the depth of the water, the flow condition will be either weir or orifice. The transition between weir and orifice occurs at approximately 7 inches depth of flow. The HEC-22 method of design for slotted pipe is recommended as the basis for grated line drain design. It should be noted that this is inlet interception/capacity design, not the carrying capacity of the product as a conduit.

Furthermore, the grated line drain has a smaller cross sectional area than slotted pipe, and therefore typically less carrying capacity.

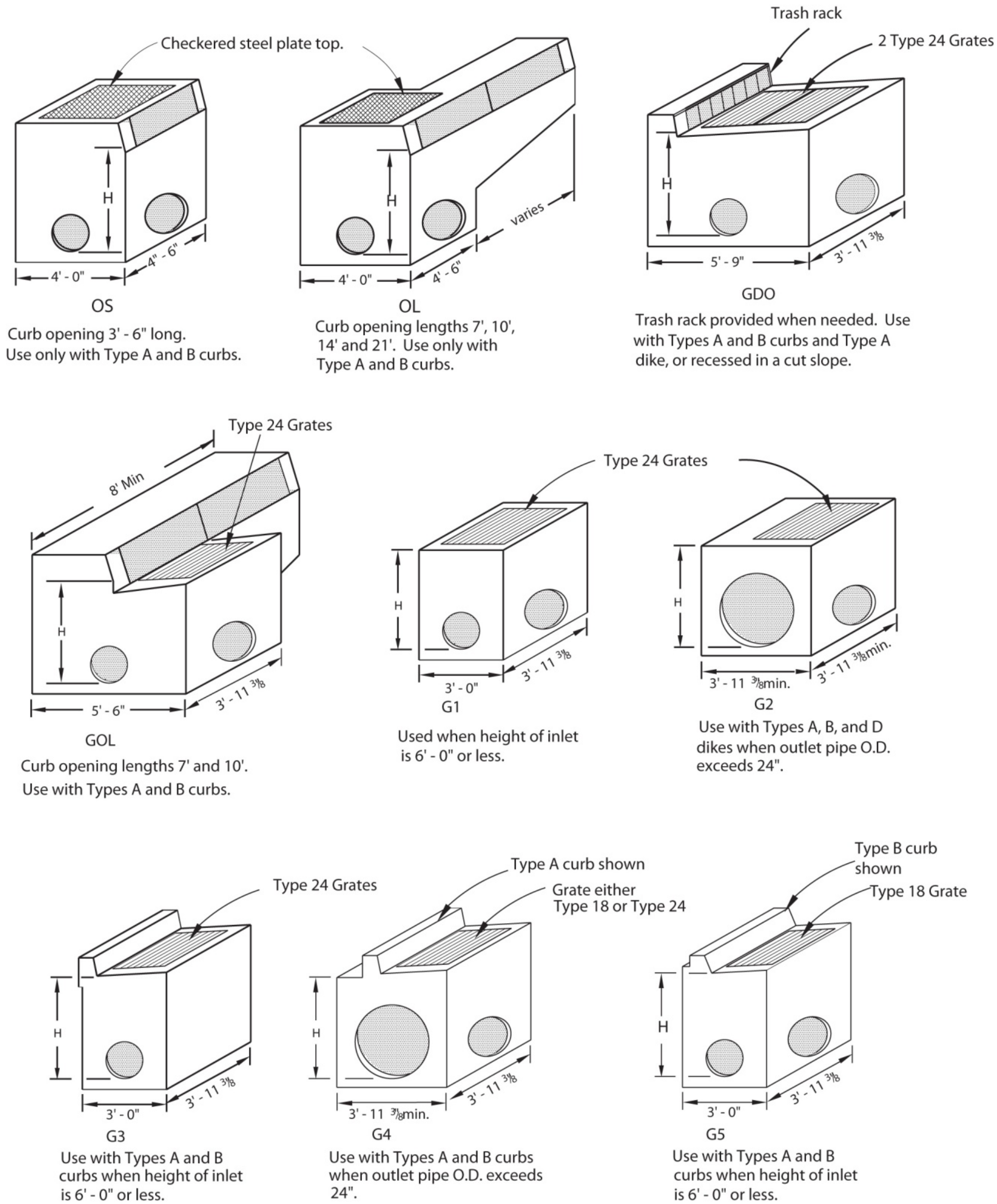
Grated line drains are recommended as an alternative to slotted pipe at locations susceptible to pipe clogging from sediments and debris. Self-cleaning velocities can usually be generated from their smooth interior surface, or if necessary by specifying the optional pre-sloped sections.

Grated line drains may also be useful where there is a potential for utility conflicts with slotted drains, which are generally installed at a greater depth.

At locations where clean out access is needed, removable grates can be specified. In areas with pedestrian traffic, special grates which meet the Americans with Disabilities Act (ADA) requirements are mandatory. This type of grate is susceptible to clogging, therefore removable grates are recommended at these locations, and they should only be specified when placement directly within the pedestrian path of travel is unavoidable.

(7) *Scuppers.* This type of inlet consists of a low, rectangular slot cut through the base of a barrier. Similar to, but smaller than curb opening inlets (See Index 837.2(1)), scuppers

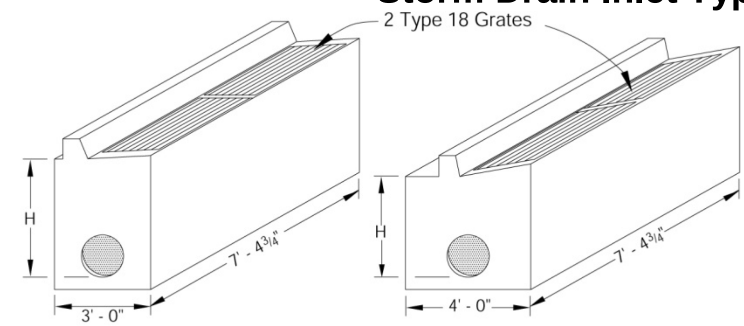
Figure 837.1
Storm Drain Inlet Types



NOTES:
1. All dimensions are outside dimensions based on 6" wall thickness.
2. For full details on uses according to type, see Index 837.2.
3. H = height of inlet.
4. See Standard Plans for Details.
5. Grates shown are not bicycle proof nor ADA compliant.

Figure 837.1

Storm Drain Inlet Types (Cont.)

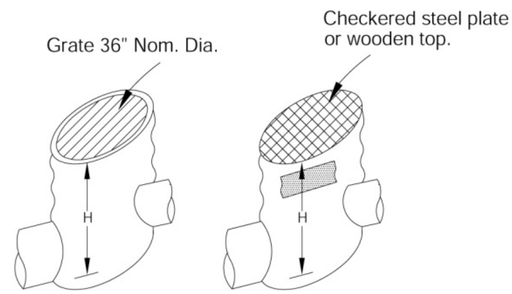


GT1

Use with Types A and B curbs when height of inlet is 6' or less.

GT2

Use with Types A and B curbs when outlet pipe O.D. exceeds 24".

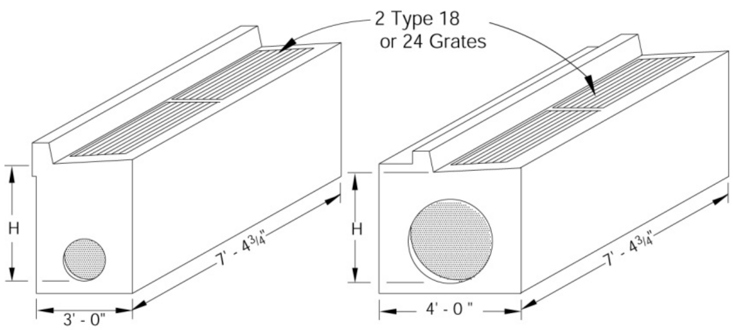


GMP

36" Diameter Metal Pipe.

OMP

36" Diameter Metal Pipe.

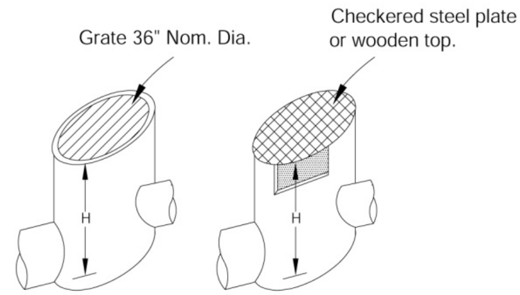


GT3

Use with Types A and B curbs when height of inlet is 6' - 0" or less.

GT4

Use with Types A and B curbs when outlet pipe O.D. excees 24".

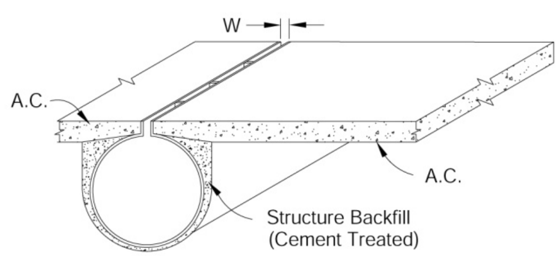


GCP

36" Diameter Concrete Pipe.

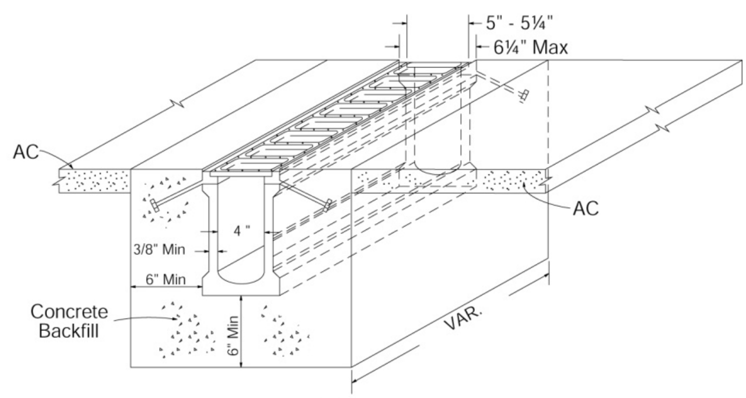
OCP

36" Diameter Concrete Pipe.



SLOTTED DRAIN INLET

12" to 24" Diameter Corrugated Metal Pipe. W = 1 3/4"



GRATED LINE DRAIN

Precast with non-integral frame

NOTES:
1. All dimensions are outside dimensions based on 6" wall thickness.
2. For full details on uses according to type, see Index 837.2.
3. H = height of inlet.
4. See Standard Plans for Details.

are prone to clogging by sediment and debris and require enhanced maintenance attention. Scupper interception efficiency decreases with increased longitudinal gradient and scupper design is not typically compatible with construction of an inlet depression. Scuppers are typically considered only when other inlet options are infeasible.

837.3 Location and Spacing

(1) *Governing Factors.* The location and spacing of inlets depend mainly on these factors:

(a) The amount of runoff,

(b) The longitudinal grade and cross slope,

(c) The location and geometrics of interchanges and at-grade intersections,

(d) Tolerable water spread, see Table 831.3,

(e) The inlet capacity,

(f) Accessibility for maintenance and inspection,

(g) Volume and movements of motor vehicles, bicycles and pedestrians,

(h) Amount of debris, and

(i) The locations of public transit stops.

(2) *Location.* There are no ready rules by which the spacing of inlets can be fixed; the most effective and economical installation should be the aim.

The following are locations where an inlet is nearly always required:

- Sag points
- Points of superelevation reversal
- Upstream of ramp gores
- Upstream and downstream of bridges – bridge drainage design procedure assumes no flow onto bridge from approach roadway, and flow off bridge to be handled by the district.
- Intersections
- Upstream of pedestrian crosswalks
- Upstream of curbed median openings

In urban areas, the volume and movements of vehicles, bicyclists, and pedestrians constitute an important control. For street or road crossings, the usual inlet location is at the intersection at the upstream end of the curb or pavement return and clear of the pedestrian crosswalk. Where the gutter flow is small and vehicular, bicycle, and pedestrian traffic are not important considerations, the flow may be carried across the intersection in a valley gutter and intercepted by an inlet placed downstream. See Index 836.2(4).

At depressed grade lines under structures, care must be taken to avoid bridge pier footings. See Index 204.6.

Safety of location for maintenance purposes is an important consideration. Wall opening inlets should not be placed where they present an obstacle to maintenance equipment and to vehicles that leave the traveled way. Grate top inlets should be installed in such locations.

Placement of inlets within the traveled way is discouraged. Inlets should typically be relocated when roadways are widened or realigned. Any proposal to leave an existing or construct a new inlet within the traveled way should be discussed with District Maintenance to verify that future access is feasible.

(3) *Spacing.* Arbitrary spacing of inlets should be avoided. The distance between inlets should be determined by a rational analysis of the factors mentioned above. Detailed procedures for determining inlet spacing are given in FHWA Hydraulic Engineering Circular No. 22, "Urban Drainage Design Manual". In a valley median, the designer should consider the effect of inlet spacing on flow velocities where the soil is susceptible to erosion. To economize on disposal facilities, inlets are often located at culverts or near roadway drainage conduits.

(4) *Inlets in Series.* Where conditions dictate the need for a series of inlets, the recommended minimum spacing should be approximately 20 feet to allow the bypass flow to return to the curb face.

837.4 Hydraulic Design

(1) *Factors Governing Inlet Capacity.* Inlet capacity is a variable which depends on:

(a) The size and geometry of the intake opening,

(b) The velocity and depth of flow and the gutter cross slope just upstream from the intake, and

(c) The amount of depression of the intake opening below the flow line of the waterway.

(2) *General Notes.*

(a) Effect of Grade Profile. The grade profile affects both the inlet location and its capacity. The gutter grade line exerts such an influence that it often dictates the choice of inlet types as well as the gutter treatment opposite the opening. See Index 831.2.

Sag vertical curves produce a flattening grade line which increases the width of flow at the bottom. To reduce ponding and possible sedimentation problems, the following measures should be considered:

- Reduce the length of vertical curve.
- Use a multiple installation consisting of one inlet at the low point and one or more inlets upstream on each side. Refer to HEC 22 for further discussion and design procedures for locating multiple inlets.

  Short sections of slotted or grated line drains on either side of the low point may be used to supplement drop inlets.

(b) Cross Slope for Curbed Gutters. Make the cross slope as steep as possible within limits stated under Index 836.2(2). This concentrates the flow against the curb and greatly increases inlet capacity.

(c) Local Depressions. Use the maximum depression consistent with site conditions; for further details see Index 837.5.

(d) Trash. The curb-opening type inlet, when the first in a series of grate inlets, may intercept trash and improve grate efficiency. In a grade sag, one trash interceptor should be used on each side of the sump.

(e) Design Water Surface Within the Inlet. The crown of the outlet pipe should be low enough to allow for pipe entrance losses plus a freeboard of 0.75 feet between the design water surface and the opening at the gutter intake. This allows sufficient margin for turbulence losses, and the effects of floating trash.

(f) Inlet Floor. The inlet floor should generally have a substantial slope toward the outlet. In a shallow drain system where conservation of head is essential, or any system where the preservation of a nonsilting velocity is necessary, the half round floor shown on the Standard Plan D74C should be used when a pipe continues through the inlet.

(g) Partial Interception. Economies may be achieved by designing inlets for partial interception with the last one or two inlets in series intercepting the remaining flow. See Hydraulic Engineering Circular No. 22.

(3) *Curb-Opening Inlets.* Gutter depressions should be used with curb-opening inlets. The standard gutter depressions for curb-opening inlets, shown on Standard Plan D78 are 0.1 foot and 0.25 foot deep.

Curb-opening inlets are most economical and effective if designed and spaced to intercept only 85 to 90 percent of the flow. This provides for an increased flow depth at the curb face.

Figure 4-11, "Comparison of Inlet Interception Capacity, Slope Variable", and Figure 4-12, "Comparison of Inlet Interception Capacity, Flow Rate Variable" of Hydraulic Engineering Circular No. 22 can be used to obtain interception capacities for various longitudinal grades, cross slopes, and gutter depressions. Charts for determining interception capacities under sump conditions are also available in HEC No. 22.

(4) *Grate Inlets.* The grate inlet interception capacity is equal to the sum of the frontal flow (flow over the grate) interception and the side flow interception. The frontal flow interception will constitute the major portion of the grate interception. In general, grate inlets will intercept all of the frontal flow until a velocity is reached at which water begins to splash over the grate. Charts provided in HEC 22 can be used to compute grate interception capacities for the various grates contained therein. Grate depressions will greatly increase inlet capacity.

The HEC 22 charts neglect the effects of debris and clogging on inlet capacity. In some localities inlet clogging from debris is extensive, while in other locations clogging is negligible. Local experience should dictate the magnitude of the clogging factor, if any, to be applied. In the absence of local experience, design clogging factors of 33 percent for freeways and 50 percent for city streets may be assumed.

Grate type inlets are most economical and effective if designed and spaced to intercept only 75 to 80 percent of the gutter flow.

(5) *Combination Inlets.*

(a) Type GO and GDO Inlet. For design purposes, only the capacity of the grates need be considered. The auxiliary curb opening, under normal conditions, offers little or no increase in capacity; but does act as a relief opening should the grate become clogged. Since the grates of Type GDO are side by side, the inlet capacity is the combined capacity of the two grates.

(b) Type GOL Inlet. The interception capacity of this inlet, a curb-opening upstream of a grate, is equal to the sum of the capacities for the two inlets except that the frontal flow and thus interception capacity of the grate is reduced by interception at the curb opening.

(6) *Pipe Drop Inlets.*

(a) Wall Opening Intake. The standard intake opening 2 feet wide and 8 inches to 12 inches deep provides a capacity of approximately 6.0 CFS when the water surface is 1 foot higher than the lip of the opening. Where the flow is from more than one direction, two or more standard openings may be provided. Higher capacity openings larger than standard may be provided but are of a special design.

(b) Grate Intake. The choice between inlets with a round grate (Types GCP and GMP) and those with a rectangular grate (Type G1) hinges largely on hydraulic efficiency. In a waterway where the greatest depth of flow is at the center, both grates are equally effective. In a waterway where the cross slope concentrates the flow on one side of the grate, the rectangular shape is preferred. For rectangular grates, the charts contained in HEC 22 can be used to compute flow intercept. Round grates (Type 36R) with 0.5 foot of depression develop a capacity of 12 CFS to 15 CFS.

837.5 Local Depressions

(1) *Purpose.* A local depression is a paved hollow in the waterway shaped to concentrate and direct the flow into the intake opening and increases the capacity of the inlet. In a gutter bordered by a curb, it is called a gutter depression.

(2) *Requirements.* Local depressions generally consist of a paved apron or transition of a shape which serves the purpose. Local depressions should meet the following requirements:

(a) Valley Medians. In medians on a grade, the depression should extend a minimum of 10 feet upstream, 6 feet downstream and 6 feet laterally, measured from the edge of the opening. In a grade sag, the depression should extend a minimum of 10 feet on all sides. No median local depression, however should be allowed to encroach on the shoulder area.

The normal depth of depression is 4 inches.

(b) Paved Gutter Flares. The local depression which adjoins the outer edge of shoulder at the entrance to overside downdrains and spillways is labeled "paved gutter flare" on Standard Plans D87-A and D87-D. The flow line approaching the inlet is depressed to increase capacity and minimize water spread on the roadbed. Within a flare length of 10 feet the gutter flow line is depressed a minimum of 6 inches at the inlet. Recommended flare lengths for various gutter flow line depression depths are given on the Standard Plans. When conditions warrant, these flare lengths may be exceeded.

Traffic safety should not be compromised for hydraulic efficiency. Any change in the shape of the paved gutter flare that will result in a depression within the shoulder area should not be made. The Type 2 entrance taper and paved gutter flare is intended for use on divided highways where gutter grades exceed 2 percent and flow is in the opposite direction of traffic.

(c) Roadside Gutter and Ditch Locations. Regardless of type of intake, the opening of a drop inlet in a roadside gutter or ditch should be depressed from 4 inches to 6 inches below the flow line of the waterway with 10 feet of paved transition upstream.

(d) Curb and Gutter Depressions. This type of depression is carefully proportioned in length, width, depth, and shape. To best preserve the design shape, construction normally is of concrete. Further requirements for curb and gutter depressions are:

- Length - As shown on Standard Plan D78.
- Width - Normally 4 feet, but for wide flows or a series of closely spaced inlets, 6 feet is authorized.
- Depth - Where traffic considerations govern, the depth commonly used is 0.1 foot. Use the maximum of 0.25 foot wherever feasible at locations where the resulting curb height would not be objectionable.

(e) Type of Pavement. Local depressions outside the roadbed are usually surfaced with asphalt concrete 0.15 foot thick.

(3) *General Notes on Design.* Except for traffic safety reasons, a local depression is to be provided at every inlet even though the waterway is unpaved. Where the size of intake opening is a question, a depression of maximum depth should be considered before deciding on a larger opening. For traffic reasons, the gutter depression should be omitted in driveways and median curb and gutter installations.

It is permissible to omit gutter depressions at sump inlets where the width of flow does not exceed design water spread.

Topic 838 - Storm Drains

838.1 General

The total drainage system which conveys runoff from roadway areas to a positive outlet including gutters, ditches, inlet structures, and pipe is generally referred to as a storm drain system. In urban areas a highway storm drain often augments an existing or proposed local drainage plan and should be compatible with the local storm drain system.

This section covers the hydraulic design of the pipe or enclosed conduit portion of a storm drain system.

838.2 Design Criteria

To adequately estimate design storm discharges for a storm drain system in urban areas involving street flooding it may be necessary to route flows by using hydrograph methods. Hydrographs are discussed under Index 816.5 and further information on hydrograph methods may be found in Chapters 6 and 7 of HDS No.2, Highway Hydrology.

838.3 Hydraulic Design

Closed conduits should be designed for the full flow condition. They may be allowed to operate under pressure, provided the hydraulic gradient is 0.75 foot or more below the intake lip of any inlet that may be affected. The energy gradient should not rise above the lip of the intake. Allowances should be made for energy losses at bends, junctions and transitions.

To determine the lowest outlet elevation for drainage systems which discharge into leveed channels or bodies of water affected by tides, consideration should be given to the possibilities of backwater. The effect of storm surges (e.g., winds and floods) should be considered in addition to the predicted tide elevation.

Normally, special studies will be required to determine the minimum discharge elevation consistent with the design discharge of the facility.

838.4 Standards

(1) *Location and Alignment.* Longitudinal storm drains are not to be placed under the traveled way of highways. Depending upon local agency criteria, storm drains under the traveled way of other streets and roads may be acceptable. A manhole or specially designed junction structure is usually provided at changes in direction or grade and at locations where two or more storm drains are joined. Refer to Index 838.5 for further discussion on manholes and junction structures.

(2) *Pipe Diameter.* The minimum pipe diameter to be used is given in Table 838.4.

(3) *Slope.* The minimum longitudinal slope should be such that when flowing half full, a self cleaning velocity of 3 feet per second is attained.

(4) *Physical Properties.* In general, the considerations which govern the selection of culvert type apply to storm drain conduits. Alternative types of materials, overfill tables and other physical factors to be considered in selecting storm drain conduit are discussed under Chapter 850.

(5) *Storage.* In developing the most economical installation, the designer should not overlook economies obtainable through the use of pipeline storage and, within allowable limits, the ponding of water in gutters, medians and interchange areas. Inlet capacity and spacing largely control surface storage in gutters and medians; inlet capacity governs in sump areas.

Table 838.4
Minimum Pipe Diameter for Storm Drain Systems

Type of Drain	Minimum Diameter (in)
Trunk Drain	18
Trunk Laterals	15[(1)]
Inlet Laterals	15[(1)]

NOTE:

(1) 18 minimum if wholly or partly under the roadbed.

Specific subjects for special consideration are:

* Bedding and Backfill. Bedding and backfill consideration are discussed under Index 829.2. Maximum height of cover tables are included in Chapter 850 and minimum thickness of cover is given in Table 856.5.

* Roughness Factor. The roughness factor, Manning's n value, generally assumes greater importance for storm drain design than it does for culverts. Suggested Manning's n values for various types of pipe materials are given in Table 852.1.

(6) *Floating Trash.* Except at pumping installations, every effort should be made to carry all floating trash through the storm drain system. Curb and wall opening inlets are well suited for this purpose. In special cases where it is necessary to exclude trash, as in pumping installations, a standard trash rack must be provided across all curb and wall openings of tributary inlets. See the Standard Plans for details.

(7) *Median Flow.* In estimating the quantity of flow in the median, consideration should be given to the effects of trash, weeds, and plantings.

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 840
SUBSURFACE DRAINAGE

Topic 841 - General

Index 841.1 - Introduction

Saturation of the structural section or underlying foundation materials is a major cause of premature pavement failures. In addition, saturation can lead to undesirable infiltration into storm drain systems and, where certain soil types are below groundwater, liquefaction can occur due to seismic forces. Subsurface drainage systems designed to rapidly remove and prevent water from reaching or affecting the roadbed are discussed in this chapter.

The solution for subsurface drainage problems often calls for a knowledge of geology and the application of soil mechanics. The Project Engineer should request assistance from Geotechnical Services in the Division of Engineering Services for projects involving cuts, sections depressed below the original ground surface, or whenever the presence of groundwater is likely. Geotechnical Services can also provide assistance related to the design of features to relieve hydrostatic pressure at bridge abutments. The designer should consider the potential for large fluctuations in groundwater levels. Wet periods after several years of drought, or changes to recharge practices can lead to considerable rises in groundwater levels.

For tunnel, structure abutments, or other structure projects which might require relief of hydrostatic pressures, contact Geotechnical Services.

The basis for design will generally be the Geotechnical Design Report. This report will include findings on subsurface conditions and recommendations for design. Refer to Topic 113 for more information on Geotechnical Design Reports.

There are many variables and uncertainties as to the actual subsurface conditions. In general, the more obvious subsurface drainage problems can be anticipated in design; the less obvious are frequently uncovered during construction. Extensive exploration and literature review may be required to obtain the design variables with reasonable accuracy.

841.2 Subsurface (Groundwater) Discharge

Groundwater, as distinguished from capillary water, is free water occurring in a zone of saturation below the ground surface. Subsurface discharge, the rate at which groundwater and infiltration water can be removed depends on the effective hydraulic head and on the permeability, depth, slope, thickness and extent of the water-bearing formation (the aquifer). The discharge can be obtained by analytical methods. Such methods, however, are usually cumbersome and unsatisfactory; field explorations will yield better results.

841.3 Preliminary Investigations

Field investigations may include:

- Soils, geological, and geophysical studies.
- Borings, pits, or trenches to find the elevation, depth, and extent of the aquifer.
- Inspection of cut slopes in the immediate vicinity.
- Measurement of groundwater discharge.

Preliminary investigations should be as thorough as possible, recognizing that further information is sometimes uncovered during construction. Where an existing road is part of new construction, the presence and origin of groundwater is often known or easily detected. Personnel responsible for maintenance of the existing road are an excellent source of such information and should be consulted. Explorations, therefore, are likely to be lesser in scope and cost than explorations for a project on new alignment. In slope stability questions, and other problems of equal importance, an extensive knowledge of subsurface conditions is required. The District should ask for the assistance of Geotechnical Services in such cases.

841.4 Exploration Notes

In general, explorations should be made during the rainy season or after the melting of snow in regions where snow cover is common. An exception would be where seepage occurs from irrigation sources.

Groundwater difficulties frequently stem from water perched on an impermeable layer some distance above the actual water table. Perched water problems can often be solved with horizontal drains. See Index 841.5.

Pumped water supply wells often give unreliable indications of the water table and such data should be used with caution.

841.5 Category of System

Depending upon the scope and complexity of the problem, an appropriate solution may require the installation of one or a combination of different types of subsurface drainage systems. The type of subsurface drainage system initially considered is usually an underdrain.

The standard underdrain is the pipe underdrain. A pipe underdrain consists of a perforated pipe near the bottom of a narrow trench lined with filter fabric and backfilled with permeable material.

Pipe underdrains are discussed in more detail under Topic 842.

"French Drains" have proven to be unreliable underdrains. A "French drain" consists of a trench backfilled with rock. They are not to be used where a permanent solution is needed. Exceptions may be made for special cases such as where depth of the underdrain and soil conditions would conflict with industrial safety regulations. Under such circumstances a design that includes a filter fabric liner and permeable material backfill, without the perforated pipe may be used.

In addition to pipe underdrains, the following special purpose categories of subsurface drains are used to intercept, collect, and discharge groundwater.

- *Structural Section and Edge Drains*. Subsurface drainage systems that are primarily designed for the rapid removal of surface water infiltration from treated or untreated pavement structural section materials are called structural section drains or more typically edge drains. A 3-inch slotted plastic pipe with 3 rows of slots is the standard for structural section drains. Refer to Chapter 650, Pavement Drainage for more information on the drainage of the pavement structural section.

- *Horizontal Drains*. Horizontal drains are 1 1/2 inch perforated or slotted pipes placed in drilled holes bored into the aquifer or water bearing formations. They are installed in cut slopes and under fills more to guard against slides by relieving hydrostatic pressure than to prevent saturation of the roadbed. They may be used in varying lengths up to 1,000 feet on grades that range from 0 to 25 percent. A collection system to remove the intercepted water from the area is generally also required.

- *Prefabricated Geocomposite Drains*. Available in sheets or rolls, geocomposite drains provide a cost effective solution to subsurface drainage behind bridge abutments, wingwalls and retaining walls. Prefabricated subsurface drainage systems consist of a plastic drain core covered on one or both sides with a filter fabric.

- *Stabilization Trenches*. This category of subsurface drainage system is constructed in swales, ravines, and under sidehill fills to stabilize water logged fill foundations. The Geotechnical Design Report should contain depth and width of trench recommendations. Stabilization trenches may be only a few feet in width requiring a backhoe or similar type of excavation equipment, or they may be large enough for earth moving equipment such as dozers and scrapers to operate. Trenches wide enough to permit the use of earth moving equipment should be considered wherever feasible. A 1:1 side slope is commonly used.

  The excavated trench, including the side slopes, is covered with a thick blanket of permeable material. One or more perforated drain pipes, usually 8 inches to 12 inches in diameter, are placed at the bottom of the trench depending on the quantity of groundwater, type of material, and area to be stabilized.

  The alignment of the trench and collector pipe are often made parallel to the highway centerline. Conditions may be such that trench alignment on a skew or with tee, wye, or herringbone configurations are a better design.

Lining the trench with filter fabric is recommended. The usual 3 feet or more thickness of permeable material may be reduced and a less expensive gradation may be specified if a filter fabric is used. Assistance in selecting filter fabric and permeable material specifications should be requested from Geotechnical Services.

- *Drainage Galleries*. Drainage galleries consist of a row or rows of closely spaced wells 36 inches to 48 inches in diameter bored with power augers to the depth required to intercept the aquifer. They are a variation of the stabilization trench principle and may afford a more cost effective solution under certain conditions.

  Drainage galleries are a viable option where the depth of the aquifer exceeds the economical or practical limits for open trench excavation. Because of potential cave-ins or slides, open trench excavation may not be practical.

  The bottom of the bored wells should be interconnected and a suitable collector and outlet system must be provided. The wells may be interconnected by belling out at the bottoms, tunneling between wells, drilled-in-place outlets, or horizontal drains.

  The wells are backfilled with permeable material. The Geotechnical Design Report should contain well spacing and depth recommendations. Assistance in selecting permeable material and other specifications pertinent to drainage galleries should be requested from Geotechnical Services.

Topic 842 - Pipe Underdrains

842.1 General

As stated under Index 841.5, the standard underdrain treatment is the perforated pipe underdrain. Pipe underdrain systems consist of a 6-inch or 8-inch diameter perforated pipe placed near the bottom of a narrow trench. The trench is usually lined with filter fabric prior to placement of the perforated pipe and permeable material backfill.

Two standard cross sections for pipe underdrains are shown on Standard Plan D102. The one with the permeable material carried to the top of the grading plane is used under paved areas. The other, with a topping of earth backfill over the permeable material, is used under unpaved areas.

842.2 Single Installations

A single pipe underdrain is commonly used in these cases:

- Along the toe of a cut slope to intercept seepage when slope stability is not a problem.
- Along the toe of a fill on the side from which groundwater originates.
- Across the roadway at the downhill end of a cut.

842.3 Multiple Installations

Multiple underdrain installations may be used in a herringbone or other effective pattern in situations such as the following:

- Under the roadway structural section when a permeable blanket is required.
- To stabilize fill foundation areas.

Refer to Table 842.4 for a guide to selecting depth and spacing of multiple pipe underdrain installations.

842.4 Design Criteria

- *Size and Length*. For pipe underdrains of 500 feet or less in length, the standard perforated pipe size is 6 inches in diameter. As a rule, the 6-inch diameter is adequate for collectors and laterals in most soils. For lengths exceeding 500 feet, the minimum diameter of pipe is 8 inches.
- *Surface Runoff*. Surface drainage should be prevented from discharging into underdrain systems.

- *Outlets*. Underdrain outlets should be provided at intervals of not more than 1,000 feet.

  Underdrain systems may be designed to discharge directly into a storm drain or culvert as long as the underdrain outlet is not subjected to hydrostatic pressures that could cause backflow damage.

- *Cleanouts*. Terminal and intermediate risers may be placed for the convenience of the maintenance forces cleaning the system. When practical, a terminal riser should be placed at the upper end of an underdrain. Intermediate cleanout risers may be placed at intervals of 500 feet and at sharp angle points greater than 10 degrees.

  The diameter of risers should be the same as the pipe underdrain. Details of underdrain risers are shown on Standard Plan D102.

- *Grade*. If possible, pipe underdrains should be placed on grades steeper than 0.5 percent. Minimum grades of 0.2 percent for laterals and 0.25 percent for mains are acceptable.

- *Depth and Spacing*. The depth of the underdrain depends on the permeability of the soil, the elevation of the water table, and the amount of drawdown needed to ensure stability. Whenever practicable, an underdrain pipe should be set in the impervious zone below the aquifer. Additionally, consideration should be given to the depth and proximity of storm drains. Typically, the underdrain should be placed at a depth sufficient to keep the storm drain above the groundwater table.

  Table 842.4 gives suggested depths and spacing of underdrains according to soil types. It is only a guide and should not be considered a substitute for field observations or local experience.

842.5 Types of Underdrain Pipe

The aim of any underdrain installation is long term effectiveness. This aim is associated with filtering ability, durability, strength, and cost of conduit, mainly in that order. In choosing between pipes of different types, the key considerations are filtering ability and durability. Pipe cost assumes secondary importance because it is a minor part of the underdrain investment.

Pipes for underdrains are perforated and may be made of steel, aluminum, polyvinyl chloride (PVC) or polyethylene, all with corrugated profiles, or smooth wall PVC. All of the listed types are acceptable for either shallow or deep burial situations. Where plastic pipe underdrains are proposed and burial depths would exceed 30 feet, the Underground Structures Unit in the Division of Engineering Services should be contacted for approval.

842.6 Design Service Life

Refer to Chapter 850 for further discussion and criteria relative to design service life of pipe materials used in underdrain installations.

Experience with underdrains has shown that they are not subject to corrosion in an environment that lacks an adequate supply of air and oxygen entrained in the water. Subsurface waters that may be inclined to be corrosive chemically do not tend to become so as long as they are not exposed to oxygen. However, subsurface water may become corrosive after it has surfaced and been exposed to oxygen. Furthermore, there is evidence that indicates there is little oxygen available in long lengths of the small diameter pipe normally used in a subsurface drainage system.

Although tests may indicate that corrosive salts are present in the soil solution, corrosion will not take place without the presence of oxygen. Therefore, when it is anticipated that the underdrain will be placed to intercept groundwater under the above conditions, it will not be necessary to allow for metal pipe corrosion.

When the above conditions do not prevail, the design service life of metal pipe is determined from pH and resistivity tests covered in California Test 643. This information is shown in the Geotechnical Design Report. The design service life of steel pipe may be increased by a bituminous coating as indicated in Table 855.2C.

The guide values contained in the tables mentioned above may be modified where field observation of existing installations dictates.

842.7 Pipe Selection

In cases where more than one material meets the foregoing requirements, alternatives should be specified on the basis of optional selection by the contractor. The selection of a single type of underdrain may be appropriate due to other related factors. This selection should be supported by complete analysis of factors and documentation placed on file in the District.

Table 842.4

Suggested Depth and Spacing of Pipe Underdrains for Various Soil Types

	Soil Composition			Drain Spacing (ft)			
Soil Class	Percent Sand	Percent Silt	Percent Clay	3 feet Deep	4 feet Deep	5 feet Deep	6 feet Deep
Clean Sand	80-100	0-20	0-20	110 - 150	150 - 200	- -	- -
Sandy Loam	50-80	0-50	0-20	50 - 100	100 - 150	- -	- -
Loam	30-50	30-50	0-20	30 - 60	40 - 80	50 - 100	60 - 120
Clay Loam	20-50	20-50	20-30	20 - 40	25 - 50	30 - 60	40 - 80
Sandy Clay	50-70	0-20	30-50	15 - 30	20 - 40	25 - 50	30 - 60
Silty Clay*	0-20	50-70	30-50	10 - 25	15 - 30	20 - 40	25 - 50
Clay*	0-50	0-50	30-100	15(max)	20(max)	25(max)	40(max)

* Drainage blankets or stabilization trenches should be considered.

CHAPTER 850
PHYSICAL STANDARDS

Topic 851 - General

Index 851.1 - Introduction

This chapter deals with the selection of drainage facility material type and sizes including pipes, pipe liners, pipe linings, drainage inlets and trench drains.

851.2 Selection of Material and Type

The choice of drainage facility material type and size is based on the following factors:

(1) *Physical and Structural Factors.* Of the many physical and structural considerations, some of the most important are:

- (a) Durability.
- (b) Headroom.
- (c) Earth Loads.
- (d) Bedding Conditions.
- (e) Conduit Rigidity.
- (f) Impact.
- (g) Leak Resistance.

(2) *Hydraulic Factors.* Hydraulic considerations involve:

- (a) Design Discharge.
- (b) Shape, slope and cross sectional area of channel.
- (c) Velocity of approach.
- (d) Outlet velocity.
- (e) Total available head.
- (f) Bedload.
- (g) Inlet and outlet conditions.
- (h) Slope.
- (i) Smoothness of conduit.
- (j) Length.

Suggested values for Manning's Roughness coefficient (n) for design purposes are given in Table 851.2 for each type of conduit. See Index 866.3 for use of Manning's formula.

Topic 852 - Pipe Materials

852.1 Reinforced Concrete Pipe (RCP)

(1) *Durability.* RCP is generally precast prior to delivery to the project site. The durability of reinforced concrete pipe can be affected by abrasive flows or acids, chlorides and sulfate in the soil and water. See Index 855.2 Abrasion, and Index 855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates.

The following measures increase the durability of reinforced concrete culverts:

- (a) Cover Over Reinforcing Steel. Additional cover over the reinforcing steel should be specified where abrasion is likely to be severe as to appreciably shorten the design service life of a concrete culvert. This extra cover is also warranted under exposure to corrosive environments, see Index 855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates. Extra cover over the reinforcing steel does not necessarily require extra wall thickness, as it may be possible to provide the additional cover and still obtain the specified D-load with standard wall thicknesses.
- (b) Increase cement content.
- (c) Reduce water content.
- (d) Invert paving/plating.

(2) *Indirect Design Strength Requirements.*

- (a) Design Standards. The “D” load strength of reinforced concrete pipe is determined by the load to produce a 0.01 inch crack under the “3-edge bearing test” called for in AASHTO Designations M 170, M 207M/M 207, and M 206M/M 206 for circular reinforced pipe, oval shaped reinforced pipe, and reinforced concrete pipe arches, respectively.
- (b) Height of Fill. See Topic 856.

Table 851.2

Manning "n" Value for Alternative Pipe Materials(1)

Type of Conduit		Recommended Design Value	"n" Value Range
Corrugated Metal Pipe (2)			
(Annular and Helical) (3)			
2⅔" x ½"	corrugation	0.025	0.022 - 0.027
3" x 1"	"	0.028	0.027 - 0.028
5" x 1"	"	0.026	0.025 - 0.026
6" x 2"	"	0.035	0.033 - 0.035
9" x 2½"	"	0.035	0.033 - 0.037
Concrete Pipe			
Pre-cast		0.012	0.011 - 0.017
Cast-in-place		0.013	0.012 - 0.017
Concrete Box		0.013	0.012 - 0.018
Plastic Pipe (HDPE and PVC)			
Smooth Interior		0.012	0.010 - 0.013
Corrugated Interior		0.022	0.020 - 0.025
Spiral Rib Metal Pipe			
¾" (W) x 1" (D) @ 11½" o/c		0.013	0.011 - 0.015
¾" (W) x ¾" (D) @ 7½" o/c		0.013	0.012 - 0.015
¾" (W) x 1" (D) @ 8½" o/c		0.013	0.012 - 0.015
Composite Steel Spiral Rib Pipe		0.012	0.011 - 0.015
Steel Pipe, Ungalvanized		0.015	--
Cast Iron Pipe		0.015	--
Clay Sewer Pipe		0.013	--
Polymer Concrete Grated Line Drain		0.011	0.010 - 0.013

Notes:

(1) Tabulated n-values apply to circular pipes flowing full except for the grated line drain. See Note 5.

(2) For lined corrugated metal pipe, a composite roughness coefficient may be computed using the procedures outlined in the HDS No. 5, Hydraulic Design of Highway Culverts.

(3) Lower n-values may be possible for helical pipe under specific flow conditions (refer to FHWA's publication Hydraulic Flow Resistance Factors for Corrugated Metal Conduits), but in general, it is recommended that the tabulated n-value be used for both annular and helical corrugated pipes.

(4) For culverts operating under inlet control, barrel roughness does not impact the headwater. For culverts operating under outlet control barrel roughness is a significant factor. See Index 825.2 Culvert Flow.

(5) Grated Line Drain details are shown in Standard Plan D98C and described under Index 837.2(6) Grated Line Drains. This type of inlet can be used as an alternative at the locations described under Index 837.2(5) Slotted Drains. The carrying capacity is less than 18-inch slotted (pipe) drains.

(3) *Shapes.* Reinforced concrete culverts are available in circular and oval shapes. Reinforced Concrete Pipe Arch (RCPA) shapes have been discontinued by West Coast manufacturers.

In general, the circular shaped is the most economical for the same cross-sectional area. Oval shapes are appropriate for areas with limited head or overfill or where these shapes are more appropriate for site conditions. A convenient reference of commercially available products and shapes is the AASHTO publication, "A Guide to Standardized Highway Drainage Products".

(4) *Non-Reinforced Concrete Pipe Option.* Non-reinforced concrete pipe may be substituted at the contractor's option for reinforced concrete pipe for all sizes 36 inches in diameter and smaller as long as it conforms to Section 65 of the Standard Specifications. Non-Reinforced concrete pipe is not affected by chlorides or stray currents and may be used in lieu of RCP in these environments without coating or the need to provide extra cover over reinforcement.

(5) *Direct Design Method - RCP.* (Contact DES - Structures Design)

852.2 Concrete Box and Arch Culverts

(1) *Box Culverts.* Single and multiple span reinforced concrete box culverts are completely detailed in the Standard Plans. For cast-in-place construction, strength classifications are shown for 10 feet and 20 feet overfills. Precast reinforced concrete box culverts require a minimum of 1 foot of overfill and are not to exceed 12 feet in span length. Special details are necessary if precast boxes are proposed as extensions for existing box culverts. Where the use of precast box culverts is applicable, the project plans should include them as an alternative to cast-in-place construction. Because the standard measurement and payment clauses for precast RCB's differ from cast-in-place construction, precast units must be identified as an alternative and the special provision must be appropriately modified.

The standard plan sheets for precast boxes show details which require them to be layed out with joints perpendicular to the centerline of the box. This is a consideration for the design engineer in situations which require stage construction and when the culvert is to be aligned on a high skew. This situation will require either a longer culvert than otherwise may have been needed, or a special design allowing for skewed joints. Prior to selecting the latter option DES - Structures Design should be consulted.

(2) *Concrete Arch Culverts.* Technical questions regarding concrete arch culverts should be directed to the Underground Structures Branch of DES - Structures Design.

(3) *Three-Sided Concrete Box Culverts* Design details for cast-in-place (CIP) construction three-sided bottomless concrete box culverts in 2-foot span increments from 12 feet to < 20 feet, inclusive, with strength classifications shown for 10 feet and 20 feet overfills are available upon request from DES - Structures Design. CIP Bottomless Culvert XS-sheets 17-050-1, 2, 3, 4 and 5 may be obtained electronically. Precast three-sided box culverts are an acceptable alternative to CIP designs, where contractors may submit such designs for approval. Both precast and CIP designs must be placed on a foundation designed specifically for the project site.

(4) *Corrosion, Abrasion, and Invert Protection.* Refer to Index 855.2 Abrasion, and Index 855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates for corrosion, abrasion and invert protection of concrete box and arch culverts.

852.3 Corrugated Steel Pipe, Steel Spiral Rib Pipe and Pipe Arches

Corrugated steel pipe, steel spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. For larger diameters, arch spans or special shapes, see Index 852.5. Corrugated steel pipe and pipe arches are available in various corrugation profiles with helical and annular corrugations. Corrugated steel spiral rib pipe is available in several helical corrugation patterns.

(1) *Hydraulics.* Annular and helical corrugated steel pipe configurations are applicable in the situations where velocity reduction is important or if a culvert is being designed with an inlet control condition. Spiral rib pipe, on the other

hand, may be more appropriate for use in stormdrain situations or if a culvert is being designed with an outlet control condition. Spiral rib pipe has a lower roughness coefficient (Manning's "n") than other corrugated metal pipe profiles.

(2) *Durability*. The anticipated maintenance-free service life of corrugated steel pipe, steel spiral rib pipe and pipe arch installations is primarily a function of the corrosivity and abrasiveness of the environment into which the pipe is placed. Corrosion potential must be determined from the pH and minimum resistivity tests covered in California Test 643. Abrasive potential must be estimated from bed material that is present and anticipated flow velocities. Refer to Index 855.1 for a discussion of maintenance-free service life and Index 855.2 Abrasion, and Index 855.3 Corrosion.

The following measures are commonly used to prolong the maintenance-free service life of steel culverts:

(a) Galvanizing. Under most conditions plain galvanizing of steel pipe is all that is needed; however, the presence of corrosive or abrasive elements may require additional protection.

- Protective Coatings - The necessity for any coating should be determined considering hydraulic conditions, local experience, possible environmental impacts, and long-term economy. Approved protective coatings are bituminous asphalt, asphalt mastic and polymeric sheet, which can be applied to the inside and/or outside of the pipe; and polyethylene for composite steel spiral ribbed pipe which is a steel spiral ribbed pipe externally pre-coated with a polymeric sheet, and internally polyethylene lined. All of these protective coatings are typically shop-applied prior to delivery to the construction site. Polymeric sheet coating provides much improved corrosion resistance over bituminous coatings and can be considered to typically allow achievement of a 50-year maintenance-free service life without need to increase thickness of the steel pipe. To ensure that a damaged coating does not lead to premature catastrophic failure, the base steel thickness for pipes that are to be coated with a polymeric sheet must be able to provide a minimum 10-year service life prior to application of the polymeric material. In addition, a bituminous lining or bituminous paving can be applied over a bituminous coating primer on the inside of the pipe for extra corrosion or abrasion protection (see Section 66 of the Standard Specifications).

Citing Section 5650 of the Fish and Game Code, the Department of Fish and Game (DFG) may restrict the use of bituminous coatings on the interior of pipes if they are to be placed in streams that flow continuously or for an extended period (more than 1 to 2 days) after a rainfall event. Their concern is that abraded particles of asphalt could enter the stream and degrade the fish habitat. Where abrasion is unlikely, DFG concerns should be minimal. DFG has indicated that they have no concerns regarding interior application of polymeric sheet coatings, even under abrasive conditions.

Where the materials report indicates that soil side corrosion is expected, a bituminous asphalt coating which is hot-dipped to cover the entire inside and outside of the pipe or an exterior application of polymeric sheet, as provided in the Standard Specifications, combined with galvanizing of steel, is usually effective in forestalling accelerated corrosion on the backfill side of the pipe. Where soil side corrosion is the only, or primary, factor leading to deterioration, the bituminous asphalt protection layer described above is typically expected to add up to 25 years of service life to an uncoated (i.e., plain galvanized) pipe. A polymeric sheet coating is typically expected to provide up to 50-years of service life to an uncoated pipe. For locations where water side corrosion and/or abrasion is of concern, protective coatings, or protective coatings with pavings, or protective coatings with linings, in combination with galvanizing will add to the culvert service life to a variable degree, depending upon site conditions and type of coating selected. Refer to Index 855.2 Abrasion, and Index 855.3 Corrosion. If hydraulic conditions at the culvert site require a lining on the inside of the pipe or a coating different than that indicated in the Standard Specifications, then the different requirements must be described in the Special Provisions.

- Extra Metal Thickness. Added service life can be achieved by adding metal thickness. However, this should only be considered after protective coatings and pavings have been considered. Since 0.052 inch thick steel culverts is the minimum steel pipe Caltrans allows, it must be limited to locations that are nonabrasive.

  See Table 855.2C for estimating the added service life that can be achieved by coatings and invert paving of steel pipes based upon abrasion resistance characteristics.

(b) Aluminized Steel (Type 2). Evaluations of aluminized steel (type 2) pipe in place for over 40 years have provided data that substantiate a design service life with respect to corrosion resistance equivalent to aluminum pipe. Therefore, for pH values between 5.5 and 8.5, and minimum resistivity values in excess of 1500 ohm-cm, 0.064 inch aluminized steel (type 2) is considered to provide a 50 year design service life. Where abrasion is of concern, aluminized steel (type 2) is considered to be roughly equivalent to galvanized steel. Bituminous coatings are not recommended for corrosion protection, but may be used in accordance with Table 855.2C for abrasion resistance. A concrete invert may also be considered where abrasion is of concern. For pH ranges outside the 5.5 and 8.5 limits or minimum resistivity values below 1500 ohm-cm, aluminized steel (type 2) should not be used. In no case should the thickness of aluminized steel (type 2) be less than the minimum structural requirements for a given diameter of galvanized steel. Refer to Index 855.2 Abrasion, and Index 855.3 Corrosion.

The AltPipe Computer Program is also available to help designers estimate service life for various corrosive/abrasive conditions. See http://www.dot.ca.gov/hq/oppd/altpipe.htm

(3) *Strength Requirements*. The strength requirements for corrugated steel pipes and pipe arches, fabricated under acceptable methods contained in the Standard Specifications, are given in Tables 856.3A, B, C, & D. For steel spiral rib pipe see Tables 856.3E, F & G.

(a) Design Standards.

- Corrugation Profiles - Corrugated steel pipe and pipe arches are available in 2⅔" x ½", 3" x 1", and 5" x 1" profiles with helical corrugations, and 2⅔" x ½" profiles with annular corrugations. Corrugated steel spiral rib pipe is available in a ¾" x ¾" x 7½" or ¾" x 1" x 11½" helical corrugation pattern. For systems requiring large diameter and/or deeper fill capacity a ¾" x 1" x 8½" helical corrugation pattern is available. Composite steel spiral rib pipe is available in a ¾" x ¾" x 7½" helical ribbed profile.

- Metal Thickness - Corrugated steel pipe and pipe arches are available in the thickness as indicated on Tables 856.3A, B, C & D. Corrugated steel spiral rib pipe is available in the thickness as indicated on Tables 856.3E, F & G. Where a maximum overfill is not listed on these tables, the pipe or arch size is not normally available in that thickness. All pipe sections provided in Table 856.3 meet handling and installation flexibility requirements of AASHTO LRFD. Composite steel spiral rib pipe is available in the thickness as indicated on Table 856.3G.

- Height of Fill - The allowable overfill heights for corrugated steel and corrugated steel spiral rib pipe and pipe arches for the various diameters or arch sizes and metal thickness are shown on Tables 856.3A, B, C, & D. For corrugated steel spiral rib pipe, overfill heights are shown on Tables 856.3E, F & G. Table 856.3G gives the allowable overfill height for composite steel spiral rib pipe.

(4) *Shapes.* Corrugated steel pipe, steel spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. For larger diameters, arch spans or special shapes, see Index 852.5.

(5) *Invert Protection.* Refer to Index 855.2 Abrasion. Invert protection should be considered for corrugated steel culverts exposed to excessive wear from abrasive flows or corrosive water. Severe abrasion usually occurs when the flow velocity exceeds 12 feet per second to 15 feet per second and contains an abrasive bedload of sufficient volume. When severe abrasion or corrosion is anticipated, special designs should be investigated and considered. Typical invert protection includes invert paving with portland cement concrete with wire mesh reinforcement, and invert lining with metal plate. The paving limits for invert linings are site specific and should be determined by field review. Additional metal thickness will increase service life. Reducing the velocity within the culvert is an effective method of preventing severe abrasion. Index 853.6 provides additional guidance on invert paving with concrete.

(6) *Spiral Rib Steel.* Galvanized steel spiral rib pipe is fabricated using sheet steel and continuous helical lock seam fabrication as used for helical corrugated metal pipe. The manufacturing complies with Section 66, "Corrugated Metal Pipe," of the Standard Specifications, except for profile and fabrication requirements. Spiral rib pipe is fabricated with either: three rectangular ribs spaced midway between seams with ribs 3/4" wide x 3/4" high at a maximum rib pitch of 7-1/2 inches, two rectangular ribs and one half-circle rib equally spaced between seams with ribs 3/4" wide x 1" high at a maximum rib pitch of 11-1/2 inches with the half-circle rib diameter spaced midway between the rectangular ribs, or two rectangular ribs equally spaced between seams with ribs 3/4" wide x 1" high at a maximum rib pitch of 8-1/2 inches.

Aluminized steel spiral rib pipe, type 2 (ASSRP) is available in the same sizes as galvanized steel spiral rib and will support the same fill heights (the aluminizing is simply a replacement coating for zinc galvanizing that allows thinner steel to be placed in certain corrosive environments. See Figure 855.3A for the acceptable pH and resistivity ranges for placement of aluminized steel pipes). Tables 856.3E, F & G give the maximum height of overfill for steel spiral rib pipe constructed under the acceptable methods contained in the Standard Specifications and essentials discussed in Index 829.2.

852.4 Corrugated Aluminum Pipe, Aluminum Spiral Rib Pipe and Pipe Arches

Corrugated aluminum pipe, aluminum spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. For larger diameters, arch spans or special shapes see Index 852.6. Corrugated aluminum pipe and pipe arches are available in various corrugation profiles with helical and annular corrugations. Helical corrugated pipe must be specified if anticipated heights of cover exceed the tabulated values for annular corrugated pipe. Non-standard pipe diameters and arch sizes are also available. Aluminum spiral rib pipe is similar to spiral rib steel and is available in several helical corrugation patterns.

(1) *Hydraulics*. Corrugated aluminum pipe comes in various corrugated profiles. Annular and helical corrugated aluminum pipe configurations are applicable in the situations where velocity reduction is important or if a culvert is being designed with an inlet control condition. Spiral rib pipe, on the other hand, may be more appropriate for use in stormdrain situations or if a culvert is being designed with

an outlet control condition. Spiral rib pipe has a lower roughness coefficient (Manning's “n”) than other corrugated metal pipe profiles.

(2) *Durability*. Aluminum culverts or stormdrains may be specified as an alternate culvert material. When a 50-year maintenance-free service life of aluminum pipe is required the pH and minimum resistivity, as determined by California Test Method 643, must be known and the following conditions met:

(a) The pH of the soil, backfill, and effluent is within the range of 5.5 and 8.5, inclusive. Bituminous coatings are not recommended for corrosion protection or abrasion resistance. However, a concrete invert lining may be considered. Abrasive potential must be estimated from bed material that is present and anticipated flow velocities. Refer to Index 855.1 for a discussion of maintenance-free service life and Index 855.2 Abrasion, and Index 855.3 Corrosion prior to selecting aluminum as an allowable alternate.

(b) The minimum resistivity of the soil, backfill, and effluent is 1500 ohm-cm or greater.

(c) Aluminum culverts should not be installed in an environment where other aluminum culverts have exhibited significant distress, such as extensive perforation or loss of invert, for whatever reason, apparent or not.

(d) Aluminum may be considered for side drains in environments having the following parameters:

- When pH is between 5.5 and 8.5 and the minimum resistivity is between 500 and 1500 ohm-cm.
- When pH is between 5.0 and 5.5 or between 8.5 and 9.0 and the minimum resistivity is greater than 1500 ohm-cm.

For these conditions, the Corrosion Technology Branch in METS should be contacted to confirm the advisability of using aluminum on specific projects.

(e) Aluminum must not be used as a section or extension of a culvert containing steel sections.

(3) *Strength Requirements.* The strength requirements for corrugated aluminum pipe and pipe arches fabricated under the acceptable methods contained in the Standard Specifications, are given in Tables 856.3H, I & J. See Table 856.3K and Table 856.3L for aluminum spiral rib pipe. Tables 856.3H through L are based on the material properties of H-32 temper aluminum. Additional cover heights can be achieved for an aluminum section when H-34 temper material is used. Contact DES-Structures Design for a special design using H-34 temper material.

(a) Design Standards.

- Corrugation Profiles - Corrugated aluminum pipe and pipe arches are available in 2⅔" x ½" and 5" x 1" profiles with helical or annular corrugations. Aluminum spiral rib pipe is available in a ¾" x ¾" x 7½" or a ¾" x 1" x 11½" helical corrugation profile.
- Metal thickness - Corrugated aluminum pipe and pipe arches are available in the thickness as indicated on Tables 856.3H, I & J. Where a maximum overfill is not listed on these tables, the pipe or pipe arch is not normally available in that thickness. All pipe sections provided in Table 856.3 meet handling and installation flexibility requirements of AASHTO LRFD. Aluminum spiral rib pipe are available in the thickness as indicated on Tables 856.3K & L.
- Height of Fill - The allowable overfill heights for corrugated aluminum pipe and pipe arches for various diameters and metal thicknesses are shown on Tables 856.3H, I & J. For aluminum spiral rib pipe, overfill heights are shown on Tables 856.3K, & L.

(4) *Shapes*. Corrugated aluminum pipe, aluminum spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on

the maximum height of cover tables. Helical corrugated pipe must be specified if anticipated heights of cover exceed the tabulated values for annular corrugated pipe.

For larger diameters, arch spans or special shapes, see Index 852.5. Non-standard pipe diameters and arch sizes are also available.

(5) *Invert Protection.* Invert protection of corrugated aluminum is not recommended.

(6) *Spiral Rib Aluminum.* Aluminum spiral rib pipe is fabricated using sheet aluminum and continuous helical lock seam fabrication as used for helical corrugated metal pipe. The manufacturing complies with Section 66, "Corrugated Metal Pipe," of the Standard Specifications, except for profile and fabrication requirements. Aluminum spiral rib pipe is fabricated with either: three rectangular ribs spaced midway between seams with ribs 3/4" wide x 3/4" high at a maximum rib pitch of 7-1/2 inches or two rectangular ribs and one half-circle rib equally spaced between seams with ribs 3/4" wide x 1" high at a maximum rib pitch of 11-1/2 inches with the half-circle rib diameter spaced midway between the rectangular ribs. Figure 855.3A should be used to determine the limitations on the use of spiral rib aluminum pipe for the various levels of pH and minimum resistivity.

852.5 Structural Metal Plate

(1) *Pipe and Arches*. Structural plate pipes and arches are available in steel and aluminum for the diameters and thickness as shown on Tables 856.3M, N, O & P.

(2) *Strength Requirements*.

(a) Design Standards.

- Corrugation Profiles - Structural plate pipe and arches are available in a 6" x 2" corrugation for steel and a 9" x 2½" corrugation profile for aluminum.
- Metal Thickness - structural plate pipe and pipe arches are available in thickness as indicated on Tables 856.3M, N, O & P.
- Height of Fill - The allowable height of cover over structural plate pipe and pipe arches for the available diameters and thickness are shown on Tables 856.3M, N, O & P.

Where a maximum overfill is not listed on these tables, the pipe or arch size is not normally available in that thickness. All pipe sections provided in Table 856.3 conform to handling and installation flexibility requirements of AASHTO LRFD. Strutting of culverts, as depicted on Standard Plan D88A, is typically necessary if the pipe is used as a vertical shaft or if the backfill around the pipe is being removed in an unbalanced manner.

(b) Basic Premise. To properly use the above mentioned tables, the designer should be aware of the premises on which the tables are based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications, the conditions of cover, and pipe or arch size required by the plans and the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.

(c) Limitations. In using the tables, the following restrictions should be kept in mind.

- The values given for each size of structural plate pipe or arch constitute the maximum height of overfill or cover over the pipe or arch for the thickness of metal and kind of corrugation.
- The thickness shown is the structural minimum. For steel pipe or pipe arches, where abrasive conditions are anticipated, additional metal thickness for the invert plate(s) or a paved invert should be provided when required to fulfill the design service life requirements. Table 855.2C may be

used. See Index 855.2 Abrasion and Tables 855.2A, 855.2D and 855.2F.

- Where needed, adequate provisions for corrosion resistance must be made to achieve the required design service life called for in the references mentioned herein.
- Tables 856.3M & P show the limit of heights of cover for structural plate arches based on the supporting soil sustaining a bearing pressure of 3 tons per square foot at the corners.

(d) Special Designs. If the height of overfill exceeds the tabular values, or if the foundation investigation reveals that the supporting soil will not develop the bearing pressure on which the overfill heights for structural plate pipe or pipe arches are based, a special design prepared by DES - Structures Design is required.

(3) *Arches.* Design details with maximum allowable overfills for structural plate arches, with cast in place concrete footings may be obtained from DES - Structures Design.

(4) *Vehicular Underpasses.* Design details with maximum allowable overfills for structural plate vehicular underpasses with spans from 12 feet 2 inches to 20 feet 4 inches, inclusive, are given in the Standard Plans. These designs are based on "factored" bearing soil pressures from 2.5 tons per square foot to 11 tons per square foot.

(5) *Special Shapes.*

(a) Long Span.

- Arch
- Low Profile Arch
- High Profile Arch

(b) Ellipse. (Text Later)

- Vertical
- Horizontal

(6) *Tunnel Liner Plate.* The primary applications for tunnel liner plate include lining large structures in need of a structural repair, or culvert installations through an existing embankment that can be constructed by conventional tunnel methods. Typically, tunnel liner plate is not used for direct burial applications where structural metal plate pipe is recommended. DES - Structures Design will prepare designs upon request. See Index 853.7 for structural repairs.

852.6 Plastic Pipe

Plastic pipe is a generic term which currently includes two independent materials; the Standard Specifications states plastic pipe shall be made of either high density polyethylene (HDPE) or polyvinyl chloride (PVC) material. See Index 852.6(2)(a) Strength Requirements for allowed materials and wall profile types.

(1) *Durability.* Caltrans standards regarding the durability of plastic pipe are based on the long term performance of its material properties. Both forms of plastic pipe culverts (HDPE and PVC) exhibit good abrasion resistance and are virtually corrosion free. See Index 855.2 Abrasion and Index 855.5 Material Susceptibility to Fire. Also, see Tables 855.2A, 855.2E and 855.2F. The primary environmental factor currently considered in limiting service life of plastic materials is ultraviolet (UV) radiation, typically from sunlight exposure. While virtually all plastic pipes contain some amount of UV protection, the level of protection is not equal. Polyvinyl chloride resins used for pipe rarely incorporate UV protection (typically Titanium Dioxide) in amounts adequate to offset long term exposure to direct sunlight. Therefore, frequent exposure (e.g., cross culverts with exposed ends) can lead to brittleness and such situations should be avoided. Conversely, testing performed to date on HDPE products conforming to specification requirements for inclusion of carbon black have exhibited adequate UV resistance. PVC pipe exposed to freezing conditions can also experience brittleness and such situations should be avoided if there is potential for impact loadings, such as maintenance equipment or heavy (3" or larger) bedload during periods of freeze. Plastic pipes can also fail from long term stress that leads to crack growth and from chemical degradation. Improvements in plastic resin specifications and testing requirements has

led to increased resistance to slow crack growth. Inclusion of anti-oxidants in the material formulation is the most common form of delaying the onset of chemical degradation, but more thorough testing and assessment protocols need to be developed to more accurately estimate long term performance characteristics and durability.

(2) *Strength Requirements.*

(a) Design Standards

- Materials - Plastic pipe shall be either Type C (corrugated exterior and interior) corrugated polyethylene pipe, Type S (corrugated exterior and smooth interior) corrugated polyethylene pipe, or corrugated polyvinyl chloride pipe.
- Height of Fill - The allowable overfill heights for plastic pipe for various diameters are shown in Tables 856.4 and 856.5.

852.7 Special Purpose Types

(1) *Smooth Steel.* Smooth steel (welded) pipe can be utilized for drainage facilities under conditions where corrugated metal or concrete pipe will not meet the structural or design service life requirements, or for certain jacked pipe operations (e.g., auger boring).

(2) *Composite Steel Spiral Rib Pipe.* Composite steel spiral rib pipe is a smooth interior pipe with efficient hydraulic characteristics. See Table 851.2.

Composite steel spiral rib pipe with its interior polyethylene liner exhibits good abrasion resistance and also resists waterside corrosion found in a typical stormdrain or culvert environment. The exterior of the pipe is protected with a polyethylene film, which offers resistance to corrosive backfills. The pipe will meet a 50-year maintenance-free service life under most conditions. See Table 856.3G for allowable height of cover.

(3) *Proprietary Pipe.* See Index 110.10 for further discussion and guidelines on the use of proprietary items.

Topic 853 - Pipe Liners and Linings for Culvert Rehabilitation

853.1 General

This topic discusses alternative pipe liner and pipe lining materials specifically intended for culvert repair and does not include materials used for Trenchless Excavation Construction (e.g., pipe jacking, pipe ramming, augur boring), joint repair, various types of grouting, or standard pipe materials that are presented elsewhere in Chapter 850 and in the Standard Plans and Standard Specifications.

Many new products and techniques have been developed that often make complete replacement with open cut as shown in the Standard Plans unnecessary. When used appropriately, these new products and techniques can benefit the Department in terms of increased mobility, cost, and safety to both the public and contractors. Design Information Bulletin 83 (DIB 83) outlines a collection of procedures that are cost-effective for their location and that will meet the needs of their particular area, supplementing Topic 853. Use the following link: http://www.dot.ca.gov/hq/oppd/dib/dibprg.htm for further information.

853.2 Caltrans Host Pipe Structural Philosophy

In general, if the host (i.e., existing) pipe cannot be made capable of sustaining design loads, it should be replaced rather than rehabilitated. This is a conservative approach and when followed eliminates the need to make a detailed evaluation of the liner's ability to effectively accept and support dead and live loads. Prior to making the decision whether or not to rehabilitate the culvert and/or which method to choose, a determination of the structural integrity of the host pipe must be made. If rehabilitation of the culvert is determined to be a feasible option, existing voids within the culvert backfill or in the base material under the existing culvert identified either by Maintenance (typically as part of their culvert management system) or already noted in the Geotechnical Design Report, should be filled with grout to re-establish its load carrying capability. Therefore, structural considerations for pipe liners are generally limited to their ability to withstand construction handling and/or grouting pressures. When a structural repair is needed, contact Underground Structures within DES – Structures Design. See Index 853.7.

853.3 Problem Identification and Coordination

Before various alternatives for liners or linings can be selected, the first step following a site investigation which may include taking soil and water samples and pipe wall thickness measurements, is to determine the actual cause of the problem. Relative to Caltrans host pipe structural philosophy, the host pipe may be in need of stabilization, rehabilitation or replacement. Further, it will need to be determined if the structure is at the end of its maintenance-free service life, whether it has been damaged by mechanical abrasion, or corrosion (or both) and if there are any changes to the hydrology or habitat (e.g. fish passage). To make these determinations, the Project Engineer should coordinate with the District Maintenance Culvert Inspection team, Hydraulics and Environmental units. Further assistance may be needed from Geotechnical Design, the Corrosion Technology Branch within DES, Underground Structures and/or Structures Maintenance within DES. Prior to a comprehensive inspection either by trained personnel or camera, it may also be necessary to first clean out the culvert. Problem identification and assessment, and coordination with Headquarters and DES, is discussed in greater detail in DIB 83. Use the following link; http://www.dot.ca.gov/hq/oppd/dib/dib83-01-7.htm#7-1-6

853.4 Alternative Pipe Liner Materials

Similar to the basic policy in Topic 857.1 for alternative pipes, when two or more liner materials meet the design service life and minimum thickness requirements for various materials that are outlined under Topic 855, as well as hydraulic requirements, the plans and specifications should provide for alternative pipe liners to allow for optional selection by the contractor. A table of allowable alternative pipe liner materials for culverts and drainage systems is included as Table 853.1A. This table also identifies the various diameter range limitations and whether annular space grouting is needed. Sliplining consists of sliding a new culvert inside an existing distressed culvert as an alternative to total replacement. See DIB No 83; http://www.dot.ca.gov/hq/oppd/dib/dib83-01-6.htm#6-1-3-1.

The plastic pipeliners listed in the notes under Table 853.1A are installed as slipliners, however, other standard pipe types that are described in Topic 852 (e.g., metal), may be equally viable as material options to be added as sliplining alternatives.

Table 853.1A
Allowable Alternative Pipe Liner Materials

Allowable Alternatives	Diameter Range (1)	Annular Space Grouting
PP (2)	15" – 120"	Yes
CIPP	8" – 96"	No
MSWPVCPLED	6" – 30"	No
SWPVCPLFD	21" – 108"	Yes

Abbreviations:

PP	–	Plastic Pipe (sliplining)
CIPP	–	Cured in Place Pipe
SWPVCPLFD	–	Spiral Wound PVC Pipe Liner (Fixed Diameter)
MSWPVCPLED	–	Machine Spiral Wound PVC Pipe Liner (Expandable Diameter)

Note:

(1) Headquarters approval needed for pipe liner diameters 60 inches or larger. Diameter range represents liners only, not Caltrans standard pipe.

(2) The designer must edit the following plastic pipeliner list within SSP 15-6.10 to suit the work:

- Type S corrugated high density polyethylene (HDPE) pipe conforming to the provisions in Section 64, “Plastic Pipe,” of the Standard Specifications; or
- Standard Dimension Ratio (SDR) 35 polyvinyl chloride (PVC) pipe conforming to the requirements in AASHTO Designation: M 278 and ASTM Designation: F 679; or
- Polyvinyl chloride (PVC) closed profile wall pipe conforming to the requirements in ASTM Designation: F 1803, F 794 (Series 46); or
- Polyvinyl chloride (PVC) dual wall corrugated pipe conforming to the requirements in ASTM Designation: F 794 (Series 46), and ASTM Designation F 949; or
- High density polyethylene (HDPE) solid wall pipe conforming to the requirements in AASHTO M 326 and ASTM Designation: F 714; or

- Large diameter high density polyethylene (HDPE) closed profile wall pipe conforming to the requirements in ASTM Designation: F 894.

Table 853.1B provides a guide for plastic pipeliner selection in abrasive conditions to achieve a 50-year maintenance-free service life.

For further information on sliplining using plastic pipe liners including available dimensions and stiffness, see DIB 83. Use the following link: http://www.dot.ca.gov/hq/oppd/dib/dib83-01-6.htm#6-1-3-1-1

853.5 Cementitious Pipe Lining

This method may be used to line corroded corrugated steel pipes ranging from 12 inches to a maximum of 36 inches diameter and involves lining an existing culvert with concrete, shotcrete or mortar using a lining machine. If the bedload is abrasive, alternative cementitious materials such as calcium aluminate mortar or geopolymer mortar may be selected from the Authorized Materials list for cementitious pipeliners. See Table 855.2F and Section 15-6.14 of the Standard Specifications for specifications. Regardless of type of cementitious material used, the resulting lining is a minimum of one inch thick when measured over the top of corrugation crests and has a smooth surface texture. As with other liners, the pipes must first be thoroughly cleaned and dried. For diameters between 12 and 24 inches, the cement mortar is applied by robot. The mortar is pumped to a head, which rotates at high speed using centrifugal force to place the mortar on the walls. A conical-shaped trowel attached to the end of the machine is used to smooth the walls. The maximum recommended length of small-diameter pipe that can be lined using this method is approximately 650 feet. Although this method will line larger diameter pipes, it is mostly appropriate for non-human entry pipes (less than 30 inches). Generally, most problems with steel pipe are limited to the lower 180 degrees, therefore, in larger diameter metal pipes where human entry is possible, invert paving may be all that is required. See Index 853.6.

853.6 Invert Paving with Concrete

(1) *Existing Corrugated Metal Pipe (CMP).* One of the most effective ways to rehabilitate corroded and severely deteriorated inverts of CMP that are large enough for human entry (with equipment) is by paving them with reinforced concrete shotcrete or authorized cementitious material. Standard Specification Section 15-6.04 includes specifications for preparing the surface of the culvert invert, installing bar reinforcement and anchorage devices, and paving the invert with concrete, shotcrete or authorized cementitious material. For most non-abrasive sites, concrete may comply with the requirements for minor concrete or shotcrete. See index 110.12 Tunnel Safety Orders. Generally, this method is feasible for pipes 48 inches in diameter and larger. If abrasion is present, see Table 855.2F for minimum material thickness of concrete or authorized material. Concrete should have a minimum compressive strength of 6,000 psi at 28 days and the aggregate source should be harder material than the streambed load and have a high durability index (consult with District Materials Branch for sampling and recommendation). The maximum grading specified (1.5 inch) for coarse aggregate may need to be modified if the concrete must be pumped. The abrasion resistance of cementitious materials is affected by both its compressive strength and hardness of the aggregate. There is a correlation between decreasing the water/cement ratio, increasing compressive strength and increasing abrasion resistance. Therefore, where abrasion is a significant factor, the lowest practicable water/cement ratios and the hardest available aggregates should be used.

Paving thickness will range from 2 inches to 13 inches depending on abrasiveness of site based on Table 855.2A, and paving limits typically vary from 90 to 120 degrees for the internal angle. See Index 855.2 and Table 855.2F. Note that in Table 855.2F cementitious concrete is not recommended for extremely abrasive conditions (Level 6 in Table 855.2A). For extremely abrasive conditions alternative materials are recommended such as abrasion resistant concrete (calcium aluminate), steel plate or adding RSP. Calcium aluminate abrasion resistant concrete or mortar may be selected from the Authorized Materials list for concrete invert paving. If hydraulically feasible, a flattened invert design may be warranted.

Table 853.1B
Guide for Plastic Pipeliner Selection in Abrasive Conditions[2] to Achieve 50 Years of Maintenance-Free Service Life

		Abrasion Level[1]		
MATERIAL		4	5	6
Type S corrugated polyethylene pipe		-	-	-
Standard Dimension Ratio (SDR) 35 PVC [3]	(46 psi)	4" – 48"	12"- 48"	36"– 48"
	(75 psi)	18" – 48"	18" – 48"	30" – 48"
	(115 psi)	18" – 48"	18" – 48"	27" – 48"
Standard Dimension Ratio (SDR) PVC[4] (AWWA C900 & C905)	SDR 41	30" – 36"	30" – 36"	-
	SDR 32.5	30" – 36"	30" – 36"	30" – 36"
	SDR 25	4" – 36"	8" – 36"	24" – 36"
	SDR 21	14" – 24"	14" – 24"	20" – 24"
	SDR 18	4" – 24"	6" – 24"	18" – 24"
	SDR 14	4" – 12"	4" – 12"	-
PVC closed profile wall (ASTM F 1803)		18" – 60"	42"– 60"	-
Corrugated PVC (ASTM F 794 & F 949)	(46 psi)	18" – 36"	-	-
	(115 psi)	15"	-	-
Standard Dimension Ratio (SDR) HDPE[3] (AASHTO M 326 and ASTM Designation F 714)	SDR 41	10" – 63"	36" – 63"	-
	SDR 32.5	8" – 63"	30" – 63"	-
	SDR 26	6" – 63"	24" – 63"	-
	SDR 21	5" – 63"	20" – 63"	54" – 63"
	SDR 17	5" – 55"	16" – 55"	42" – 55"
	SDR 15.5	5" – 48"	14" – 48"	42" – 48"
	SDR 13.5	5" – 42"	12" – 42"	34" – 42"
	SDR 11	5" – 36"	10" – 36"	28" - 36"
	SDR 9	5" – 24"	8" – 24"	22"
Polyethylene (PE) large diameter profile wall sewer and drain pipe (ASTM F 894)	RSC[5] 160	18" – 120"	120"	-
	RSC[5] 250	33" – 108"	96" – 108"	-

NOTES:

(1) See Tables 855.2A and 855.2F for Abrasion Level Descriptions and minimum thickness.

(2) No restrictions for Abrasion Levels 1 through 3.

(3) Measured pipe designated SDR is measured to outside diameter.

(4) Measured to inside diameter.

(5) RSC = Ring Stiffness Class

Consult the District Hydraulic Branch for a recommendation.

Where there is significant loss of the pipe invert, it may be necessary to tie the concrete to more structurally sound portions of the pipe wall in order to transfer compressive thrust of culvert walls into the invert slab to create a "mechanical" connection using welding studs, angle iron or by other means. When a mechanical connection is used, paving limits may vary up to 180 degrees for the internal angle. These types of repairs should be treated as a special design and consultation with the Headquarters Office of Highway Drainage Design within the Division of Design and the Underground Structures unit of Structures Design within the Division of Engineering Services (DES) is advised. Depending on the size of the culvert being paved, pipes with significant invert loss often also have a significant loss of structural backfill with voids present. Where large voids are present, consultation with Geotechnical Services within the Division of Engineering Services (DES) is advised to develop a grouting plan.

See DIB 83 for some invert paving case studies using the following link: http://www.dot.ca.gov/hq/oppd/dib/dib83-01-12.htm#h

(2) *Existing RCB and RCP*. For existing reinforced concrete boxes (RCB) and reinforced concrete pipes (RCP) with worn inverts and exposed reinforcing steel (generally from abrasive bedloads), the same paving thickness considerations outlined under Index 853.6(1) will apply. However, depending on the structural condition, the existing steel reinforcement may need to be augmented. Consultation with Structures Maintenance and Underground Structures within DES is recommended.

(3) *Existing Plastic Pipe*. Generally, concrete invert paving is not feasible for plastic pipes because the cement will not adhere to plastic. However, it may be possible to create a "mechanical" connection by other means but these types of repairs should be treated as a special design and consultation with the Headquarters Office of Highway Drainage Design within the Division of Design and the Underground Structures unit of Structures Design within the Division of Engineering Services (DES) is advised.

853.7 Structural Repairs with Steel Tunnel Liner Plate

Cracks in RCP greater than 0.1 inch in width and flexible metal pipes with deflections beyond 10 – 12 percent may indicate a serious condition. When replacement is not an option for existing human entry pipes in need of structural repair, an inspection by Structures Maintenance and a structural analysis by Underground Structures within DES are recommended. Further assistance may be needed from Geotechnical Design and/or the Corrosion Unit within DES.

Two flange or four flange steel tunnel liner plate can be specially designed by Underground Structures within DES as a structural repair to accommodate all live and dead loads. The flange plate lap joints facilitate internal bolt connections (structural metal plate requires access to both sides). After the rings have been installed, the annular space between the liner plates and the host pipe is grouted.

Topic 854 - Pipe Connections

854.1 Basic Policy

The Standard Specifications set forth general performance requirements for transverse field joints in all types of culvert and drainage pipe used for highway construction.

Table 857.2 indicates the alternative types of joints that are to be specified for different arch and circular pipe installations with regard to joint strength. The two joint strength types specified for culvert and drainage systems are identified as "standard" and "positive."

(1) *Joint Strength.* Joint strength is to be designated on the culvert list.

(a) Standard Joints. The "standard" joint is usually for pipes or arches not subject to large soil movement or disjointing forces. These "standard" joints are satisfactory for ordinarily installations, where tongue and groove or simple slip type joints are typically used. The "standard" joint type is generally adequate for underdrains.

(b) Positive Joints. "Positive" joints are for more adverse conditions such as the need to withstand soil movements or resist disjointing forces. Examples of these conditions are steep slopes, sharp curves, and poor foundation conditions. See Index 829.2 for additional discussion. "Positive" joints should always be designated on the culvert list for siphon installations.

(c) Downdrain Joints. Pipe "downdrain" joints are designed to withstand high velocity flows, and to prevent leaking and disjointing that could cause failure.

(d) Joint Strength Properties. A description of the specified joint strength properties tabulated in Section 61 "Culvert and Drainage Pipe Joints" of the Standard Specifications is as follows:

- Shear Strength. The shear strength required of the joint is expressed as a percentage of the calculated shear strength of the pipe at a transverse section remote from the joint. All joints, including any connections must be capable of transferring the required shear across the joint.

- Moment Strength. The moment strength required of the joint is expressed as a percent of the calculated moment capacity of the pipe on a transverse section remote from the joint.

- Tensile Strength. The tensile strength is that which resist the longitudinal force which tends to separate (disjoint) adjacent pipe sections.

- Joint Overlap.

  Integral Preformed Joint. The Joint overlap is the amount of protection of one culvert barrel into the adjacent culvert barrel by the amount specified for the size of pipe designated. The amount of required overlap will vary based on several factors (material type, diameter, etc.) and is designated on the Standard Plans and/or Standard Specifications.

  Any part of an installed joint that has less than ¼ inch overlap will be considered disjointed. Whenever the plans require that the culvert be constructed on a curve, specially manufactured sections of culvert will be required if the design joint cannot meet the minimum ¼ inch overlap requirement after the culvert section is placed on the specified curve.

  - Sleeve Joints. The joint overlap is the minimum sleeve width (typically defined by the width of a coupling band) required to engage both the culvert barrels which are abutted to each other.

(2) Joint Leakage. The ability of a pipe joint to prevent the passage of either soil particles or water defines its soiltightness or watertightness. These terms are relative and do not mean that a joint will be able to completely stop the movement of soil or water under all conditions. Any pipe joint that allows significant soil migration (piping) will ultimately cause damage to the embankment, the roadway, or the pipe itself. Therefore, site conditions, such as soil particle size, presence of groundwater, potential for pressure flow, etc., must be evaluated to determine the appropriate joint requirement. Other than solvent or fusion welded joints, almost all joints can exhibit some amount of leakage. Joint performance is typically defined by maximum allowable opening size in the joint itself or by the ability to pass a standardized pressure test. The following criteria should be used, with the allowable joint type(s) indicated on the project plans:

- Normal Joint. Many pipe joint systems are not defined as either soiltight or watertight. However, for the majority of applications, such as culverts or storm drains placed in well graded backfill and surrounding soils containing a minimum of fines; no potential for groundwater contact; limited internal pressure, hydraulic grade line below the pavement grade, etc., this type of joint is acceptable. All currently accepted joint types will meet or exceed "Normal Joint" requirements. The following non-gasketed joint types should not be used beyond the "Normal Joint" criteria range:

CMP	-Annular -Hat -Helical -Hugger -2-piece Integral Flange -Universal
PLASTIC	-Split Coupler -Bell/Spigot

- Soiltight Joint. This category includes those joints which would provide an enhanced level of security against leakage and soil migration over the normal joint. One definition of a soiltight joint is contained in Section 26.4.2.4(e) of the AASHTO Standard Specifications for Highway Bridges. In part, this specification requires that if the size of the opening through which soil might migrate exceeds 1/8 inch, the length of the channel (length of path along which the soil particle must travel, i.e., the coupling length) must exceed 4 times the size of the opening. Alternatively, AASHTO allows the joint to pass a hydrostatic test (subjected to approx. 4.6 feet of head) without leaking to be considered soiltight. Typical pipe joints that can meet this criteria are:

RCP and NRCP	-Flared Bell -Flushed Bell -Steel Joint-Flush Bell -Single or Double Offset Design (Flared or Flushed Bell) -Double Gasket -Tongue and Groove* -Self-Centering T & G*
CMP and SSRP	-Annular w/gasket -Hat w/gasket -Helical w/gasket -Hugger w/gasket -2-piece Int. Fl. w/gasket -Universal w/gasket
CSSRP	-Cuffed end w/gasket
PLASTIC	-Split Coupler w/gasket (premium) -Bell/Spigot w/gasket

* Where substantial differential settlement is anticipated, would only meet Normal Joint criteria.

Where soil migration is of concern, but leakage rate is not, a soiltight joint can be achieved in most situations by external wrapping of the joint area with filter fabric (see Index 831.4). Joints listed under both the normal joint and soiltight joint categories, with a filter fabric wrap, would be suitable in these conditions and would not require a gasket or sealant. In many cases, fabric wrapping can be less expensive than a rubber gasket or other joint sealant. Coordination with the District Materials Unit is advised to ensure that the class of filter fabric will withstand construction handling and screen fine soil particles from migrating through the joint.

- Watertight Joint. Watertight joints are specified when the potential for soil erosion or infiltration/exfiltration must be restricted, such as for downdrains, culverts in groundwater zones, etc. Watertight joint requirements are typically met by the use of rubber gasket materials as indicated in the Standard Specifications. The watertight certification test described in Standard Specification Section 61 requires that no leakage occur when a joint is tested for a period of 10 minutes while subjected to a head of 10 feet over the crown of the pipe. This is a test that is typically performed in a laboratory under optimal conditions not typical of those found in the field. Where an assurance of water tightness is needed, a field test should be specified. Designers should be aware that field tests can be relatively expensive, and should only be required if such assurance is critical. A field leakage rate in the range of 700 gallons to 1,000 gallons per inch of nominal diameter per mile of pipe length per day, with a hydrostatic head of 6 feet above the crown of the pipe, is not unusual for joints that pass the watertight certification test, and is sufficiently watertight for well graded, quality backfill conditions. Where conditions are more sensitive, a lower rate should be specified. Rates below 50 to 100 gallons per inch per mile per day are difficult to achieve and would rarely be necessary. For example, sanitary sewers are rarely required to have leakage rates below 200 gallons per inch per mile per day, even though they have stringent health and environmental restrictions. Field hydrostatic tests are typically conducted over a period of 24 hours or more to establish a valid leakage rate. Designers should also be aware that non-circular pipe shapes (CMP pipe arches, RCP oval shapes, etc.) should not be considered watertight even with the use of rubber gaskets or other sealants due to the lack of uniform compression around the periphery of the joint. Additionally, watertight joints specified for pressure pipe or siphon applications must meet the requirements indicated in Standard Specification Sections 65 and 66. Pipe joints that meet Standard Specification Section 61 water-tightness performance criteria are:

RCP and NRCP	-Flared Bell -Flushed Bell -Steel Joint-Flush Bell -Single or Double Offset Design (Flared or Flushed Bell) -Double Gasket
CMP and SSRP	-Hugger Bands (H-10, 12) w/gasket and double bolt bar -Annular Band w/gasket -Two Piece Integral Flange w/sleeve-type gasket*
PLASTIC	-Bell/Spigot w/gasket

* Acceptable as a watertight pipe only in downdrain applications and in 6, 8 and 10 inch diameters. Factory applied sleeve-type gaskets are to be used instead of O-ring or other sealants.

Table 854.1 provides information to help the designer select the proper joint under most conditions.

Topic 855 - Design Service Life

855.1 Basic Concepts

The prediction of design service life of drainage facilities is difficult because of the large number of variables, continuing changes in materials, wide range of environments, and use of various protective coatings. The design service life of a drainage facility is defined as the expected maintenance-free service period of each installation. After this period, it is anticipated major will be needed for the facility to perform as originally designed for further periods.

For all metal pipes and arches that are listed in Table 857.2, maintenance-free service period, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of perforation at any location on the culvert (See Figures 855.3A, 855.3B, and Tables 855.2D and 855.2F). AltPipe can be used to estimate service life of all circular metal pipe. See Index 857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe.

For reinforced concrete pipe (RCP), box (RCB) and arch (RCA) culverts, maintenance-free service period, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of exposed reinforcement at any point on the culvert. AltPipe can be used to estimate service life of reinforced concrete pipe (RCP), but not RCB, RCA or NRCP. See Index 857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe.

For non-reinforced concrete pipe culverts (NRCP), maintenance-free service period, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of perforation or major cracking with soil loss at any point on the culvert.

For plastic pipe, maintenance-free service period, with respect to corrosion, abrasion, and long term structural performance, is the number of years from installation until the deterioration reaches the point

Table 854.1
Joint Leakage Selection Criteria

JOINT TYPE ⇒ ⇓ SITE CONDITIONS	"NORMAL" JOINT	"SOIL TIGHT" JOINT	"WATER TIGHT" JOINT
SOIL FACTORS			
Limited potential for soil migration (e.g., gravel, medium to coarse sands, cohesive soil)	X	X	X
Moderate potential for soil migration (e.g., fine sands, silts)	X(1)	X	X
High potential for soil migration (e.g., very fine sands, silts of limited cohesion)		X(1)	X(1)
INFILTRATION / EXFILTRATION			
No concern over either infiltration or exfiltration.	X	X	X
Infiltration or exfiltration not permitted (e.g., potential to contaminate groundwater, contaminated plume could infiltrate)			X(2)
HYDROSTATIC POTENTIAL			
Installation will rarely flow full. No contact with groundwater.	X	X	X
Installation will occasionally flow full. Internal head no more than 10 feet over crown. No potential groundwater contact.		X	X
Installation may or may not flow full. Internal head no more than 10 feet over crown. May contact groundwater.			X
Possible hydrostatic head (internal or external) greater than 10 feet, but less than 25 ft(3).			X(2)

Notes:
"X" indicates that joint type is acceptable in this application. The designer should specify the most cost-effective option.
(1) Designer should specify filter fabric wrap at joint. See Index 831.4.
(2) Designer should consider specifying field watertightness test.
(3) Pipe subjected to hydrostatic heads greater than 25 ft should have joints designed specifically for pressure applications.

of perforation at any location on the culvert or until the pipe material has lost structural load carrying capacity typically represented by wall buckling or excessive deflection/deformation. AltPipe can be used to estimate service life of all plastic pipe. See Index 857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe. All types of culverts are subject to deterioration from corrosion, or abrasion, or material degradation.

Corrosion may result from active elements in the soil, water and/or atmosphere. Abrasion is a result of mechanical wear and depends upon the frequency, duration and velocity of flow, and the amount and character of bedload. Material degradation may result from material quality, UV exposure, or long term material structural performance.

To assure that the maintenance-free service period is achieved, alternative metal pipe may require added thickness and/or protective coatings. Concrete pipe may require extra thickness of concrete cover over the steel reinforcement, high density concrete, using supplementary cementitious materials, epoxy coated reinforcing steel, and/or protective coatings. Means for estimating the maintenance-free service life of pipe, and techniques for extending the useful life of pipe materials are discussed in more detail in Topic 852.

The design service life for drainage facilities for all projects should be as follows:

(1) *Culverts, Drainage Systems, and Side Drains.*

- (a) Roadbed widths greater than 28 feet - 50 years.
- (b) Greater than 10 feet of cover - 50 years.
- (c) Roadbed widths 28 feet or less and with less than 10 feet of cover - 25 years.
- (d) Installations under interim alignment - 25 years.

(2) *Overside Drains.*

- (a) Buried more than 3 feet- 50 years.
- (b) All other conditions, such as on the surface of fill slopes - 25 years.

(3) *Subsurface Drains.*

- (a) Underdrains within roadbed - 50 years.
- (b) Underdrains outside of roadbed - 25 years.
- (c) Stabilization trench drains - 50 years.

In case of conflict in the design service life requirements between the above controls, the highest design service life is required except for those cases of interim alignment with more than 10 feet of cover. For temporary construction, a lesser design service life than that shown above is acceptable.

Where the above indicates a minimum design service life of 25 years, 50 years may be used. For example an anticipated change in traffic conditions or when the highway is considered to be on permanent alignment may warrant the higher design service life.

855.2 Abrasion

All types of pipe material are subject to abrasion and can experience structural failure around the pipe invert if not adequately protected. Abrasion is the wearing away of pipe material by water carrying sands, gravels and rocks (bed load) and is dependent upon size, shape, hardness and volume of bed load in conjunction with volume, velocity, duration and frequency of stream flow in the culvert. For example, at independent sites with a similar velocity range, bedloads consisting of small and round particles will have a lower abrasion potential than those with large and angular particles such as shattered or crushed rocks. Given different sites with similar flow velocities and particle size, studies have shown the angularity and/or volume of the material may have a significant impact to the abrasion potential of the site. Likewise, two sites with similar site characteristics, but different hydrologic characteristics, i.e., volume, duration and frequency of stream flow in the culvert, will probably also have different abrasion levels.

In Table 855.2A six abrasion levels have been defined to assist the designer in quantifying the abrasion potential of a site. The designer is encouraged to use the guidelines provided in Table 855.2A in conjunction with Table 855.2B “Bed Materials Moved by Various Flow Depths and Velocities” and the abrasion history of a site (if available) to achieve the required service life for a pipe, coating or invert lining material. Sampling of the streambed materials generally is not necessary, but visual examination and documentation of the size, shape and volume of abrasive materials in the streambed and estimating the average stream slope will provide the designer data needed to determine the expected level of abrasion. Where an existing culvert is in place, the condition of the invert and estimated combined wear rate due to abrasion and corrosion based on remaining pipe thickness measurements or if it is known approximately when first perforation occurred (steel pipe only), should always be used first. Figure 855.3B should be used to estimate the expected loss due to corrosion for steel pipe.

The descriptions of abrasion levels in Table 855.2A are intended to serve as general guidance only, and not all of the criteria listed for a particular abrasion level need to be present to justify defining a site at that level. For example, the use of one of the three lower abrasion levels in lieu of one of the upper three abrasion levels is encouraged where there are minor bedload volumes, regardless of the gradation. See Figure 855.1.

Table 855.2C constitutes a guide for estimating the added service life that can be achieved by coatings and invert paving of steel pipes based upon abrasion resistance characteristics. However, the table does not quantify added service life of coatings and paving of steel pipe based upon corrosion protection. In heavily abrasive situations, concrete inverts or other lining alternatives outlined in Table 855.2A should be considered. The guide values for years of added service life should be modified where field observations of existing installations show that other values are more accurate. The designer should be aware of the following limitations when using Table 855.2C:

- Channel Materials: If there is no existing culvert, it may be assumed that the channel is potentially abrasive to culvert if sand and/or rocks are present. Presence of silt, clay or heavy vegetation may indicate a non-abrasive flow.
- Flow velocities: The velocities indicated in the table should be compared to those generated by the 2-5 year return frequency flood.
- The abrasion levels represent all six abrasion levels presented in Table 855.2A however, levels 2 and 3 have been combined.

Figure 855.1
Minor Bedload Volume

Large, round bedload (top) and RCP with minimal wear and minor bedload volume with moderate to high velocity.

Table 855.2D constitutes a guide for anticipated wear (in mils/year) to metal pipe by abrasive channel materials. No additional abrasion wear is anticipated for steel for the lower three abrasion levels defined in Table 855.2A, because it is assumed that there is some degree of abrasion incorporated within California Test 643 and Figure 855.3B. Figure 855.3B, "Chart for Estimating Years to Perforation of Steel Culverts," is part of a Standard California Department of Transportation Test Method derived from highway culvert investigations. This chart alone is not used for determining service life because it does not consider the effects of abrasion or overfill; it is for estimating the years to the first corrosion perforation of the wall or invert of the CSP. Additional gauge thickness or invert protection may be needed if the thickness for structural requirements (i.e., for overfill) is inadequate for abrasion potential.

Table 855.2E indicates relative abrasion resistance properties of pipe and lining materials and summarizes the findings from "Evaluations of Abrasion Resistance of Pipe and Pipe Lining Materials Final Report FHWA /CA/TL-CA01-0173 (2007)". This report may be viewed at the following web address: http://www.dot.ca.gov/new/tech/researchreports/reports/2007/evaluation_of_abrasion_resistance_final_report.pdf. See Figure 855.2.

Figure 855.2
Abrasion Test Panels

Various culvert material test panels shown in Figure 855.2 after 1 year of wear at site with moderate to severe abrasion (velocities generally exceed 13 ft/s with heavy bedload).

Table 855.2F is based on Tables 855.2D and 855.2E and constitutes a guide for selecting the minimum material thickness of abrasive resistant invert protection for various materials to achieve 50 years of maintenance-free service life.

Structural metal plate pipe and arches provide a viable option for large diameter pipes (60 inches or larger) in abrasive environments because increased thickness can be specified for the lower 90 degrees or invert plates. If the thickness for structural requirements is inadequate for abrasion potential, it is recommended to apply the increased thickness to the lower 90 degrees of the pipe only. Arches, which have a relatively larger invert area than circular pipe, generally will provide a lower abrasion potential from bedload being less concentrated.

Under similar conditions, aluminum culverts will abrade between one and a half to three times faster than steel culverts. Therefore, aluminum culverts are not recommended where abrasive materials are present, and where flow velocities would encourage abrasion to occur. Culvert flow velocities that frequently exceed 5 feet per second where abrasive materials are present should be carefully evaluated prior to selecting aluminum as an allowable alternate. In a corrosive environment, Aluminum may display less abrasive wear than steel depending on the volume, velocity, size, shape, hardness and rock impact energy of the bed load. However, if it is deemed necessary to place aluminum pipe in abrasion levels 4 through 6 in Table 855.2C, contact Headquarters Office of State Highway Drainage Design for assistance.

Aluminized Steel (Type 2) can be considered equivalent to galvanized steel for abrasion resistance and therefore does not have the same limitations as aluminum in abrasive environments.

Concrete pipes typically counter abrasion through increased minimum thickness over the steel reinforcement, i.e., by adding additional sacrificial material. See Table 855.2F. However, there are significantly fewer limitations involved in increasing the invert thickness of RCB in the field verses increasing minimum thickness over the steel reinforcement of RCP in the plant. Therefore, RCP is typically not recommended in abrasive flows greater than 10 feet per second but may be considered for higher velocities if the bedload is insignificant (e.g. storm drain systems and most.

Table 855.2A
Abrasion Levels and Materials

Abrasion Level	General Site Characteristics	Allowable Pipe Materials and Lining Alternatives
Level 1	• Bedloads of silts and clays or clear water with virtually no abrasive bed load. No velocity limitation.	All pipe materials listed in Table 857.2 allowable for this level. No abrasive resistant protective coatings listed in Table 855.2C needed for metal pipe.
Level 2	• Moderate bed loads of sand or gravel • Velocities ≥ 1 ft/s and ≤ 5 ft/s (See Note 1)	All allowable pipe materials listed in Table 857.2 with the following considerations: • Generally, no abrasive resistant protective coatings needed for steel pipe. • Polymeric, or bituminous coating or an additional gauge thickness of metal pipe may be specified if existing pipes in the same vicinity have demonstrated susceptibility to abrasion and thickness for structural requirements is inadequate for abrasion potential.
Level 3	• Moderate bed load volumes of sands, gravels and small cobbles. • Velocities > 5 ft/s and ≤ 8 ft/s (See Note 1)	All allowable pipe materials listed in Table 857.2 with the following considerations: • Steel pipe may need one of the abrasive resistant protective coatings listed in Table 855.2C or additional gauge thickness if existing pipes in the same vicinity have demonstrated susceptibility to abrasion and thickness for structural requirements is inadequate for abrasion potential. • Aluminum pipe may require additional gauge thickness for abrasion if thickness for structural requirements is inadequate for abrasion potential. • Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (equivalent to galv. Steel) where pH < 6.5 and resistivity < 20,000. Lining alternatives: • PVC, • Corrugated or Solid Wall HDPE, • CIPP

Note:

(1) If bed load volumes are minimal, a 50% increase in velocity is permitted.

Table 855.2A
Abrasion Levels and Materials (Con't)

Abrasion Level	General Site Characteristics	Allowable Pipe Materials and Lining Alternatives
Level 4	• Moderate bed load volumes of angular sands, gravels, and/or small cobbles/rocks. (See Note 1) • Velocities > 8 ft/s and $\leq$ 12 ft/s	All allowable pipe materials listed in Table 857.2 with the following considerations: • Steel pipe will typically need one of the abrasive resistant protective coatings listed in Table 855.2C or may need additional gauge thickness if thickness for structural requirements is inadequate for abrasion potential. • Aluminum pipe not recommended. • Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (wear rate equivalent to galv. steel) where pH < 6.5 and resistivity < 20,000 if thickness for structural requirements is inadequate for abrasion potential. • Increase concrete cover over reinforcing steel for RCB (invert only). RCP generally not recommended. • Corrugated HDPE (Type S) limited to $\geq$ 48" min. diameter. Corrugated HDPE Type C not recommended. • Corrugated PVC limited to $\geq$ 18" min. diameter Lining alternatives: • Closed profile or SDR 35 PVC (corrugated and ribbed PVC limited to $\geq$ 18" min. diameter. • SDR HDPE • CIPP (min. thickness for abrasion specified) • Concrete and authorized cementitious pipeliners and invert paving. See Table 855.2F.

Note:

(1) For minor bed load volumes, use Level 3.

Table 855.2A
Abrasion Levels and Materials (Con't)

Abrasion Level	General Site Characteristics	Allowable Pipe Materials and Lining Alternatives
Level 5	• Moderate bed load volumes of angular sands and gravel or rock (See Note 1). • Velocities > 12 ft/s and ≤ 15 ft/s	• Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (wear rate equivalent to galv. steel) where pH < 6.5 and resistivity < 20,000 if thickness for structural requirements is inadequate for abrasion potential. • For steel pipe invert lining additional gauge thickness is recommended if thickness for structural requirements is inadequate for abrasion potential. See lining alternatives below. • Increase concrete cover over reinforcing steel for RCB (invert only). RCP generally not recommended Lining alternatives: • Closed profile (≥ 42 in) or SDR 35 PVC (PVC liners not recommended when freezing conditions are often encountered and cobbles or rocks are present) • SDR HDPE • CIPP (with min. thickness for abrasion specified) • Concrete and authorized cementitious pipeliners and invert paving. See Table 855.2F.

Note:

(1) For minor bed load volumes, use Level 3.

Table 855.2A
Abrasion Levels and Materials (Con't)

Abrasion Level	General Site Characteristics	Allowable Pipe Materials and Lining Alternatives
Level 6	• Moderate bed load volumes of angular sands and gravel or rock (See Note 1). • Velocities > 15 ft/s and ≤ 20 ft/s or • Heavy bed load volumes of angular sands and gravel or rock (See Note 1). • Velocities > 12 ft/s	• Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (wear rate equivalent to galv. steel) where pH < 5.5 and resistivity < 20,000. • None of the abrasive resistant protective coatings listed in Table 855.2C are recommended for protecting steel pipe. • Invert lining and additional gauge thickness is recommended. See lining alternatives below. • Corrugated HDPE not recommended. Corrugated and closed profile PVC pipe not recommended. • RCP not recommended. Increase concrete cover over reinforcing steel recommended for RCB (invert only) for velocities up to 15 ft/s. RCB not recommended for velocities greater than 15 ft/s unless invert lining is placed (see lining alternatives below). Lining/replacement alternatives: • ≥ 27 in SDR 35 PVC (PVC liners not recommended when freezing conditions are often encountered and cobbles or rocks are present) or HDPE SDR (minimum wall thickness 2.5") • CIPP (with min. thickness for abrasion specified), • Concrete with embedded aggregate (e.g. cobbles or RSP (facing)): (for all bed load sizes a larger, harder aggregate than the bed load, decreased water cement ratio and an increased concrete compressive strength should be specified). • Alternative invert linings may include steel plate, rails or concreted RSP, and abrasion resistant concrete (Calcium Aluminate). See authorized cementitious pipeliners and invert paving in Table 855.2F. • For new/replacement construction, consider "bottomless" structures.

Note:

(1) For minor bed load volumes, use Level 3.

Table 855.2B
Bed Materials Moved by Various Flow Depths and Velocities

Bed Material	Grain Dimensions (inches)	Approximate Nonscour Velocities (feet per second)			
		Mean Depth (feet)			
		1.3	3.3	6.6	9.8
Boulders	more than 10	15.1	16.7	19.0	20.3
Large cobbles	10 – 5	11.8	13.4	15.4	16.4
Small cobbles	5 – 2.5	7.5	8.9	10.2	11.2
Very coarse gravel	2.5 – 1.25	5.2	6.2	7.2	8.2
Coarse gravel	1.25 – 0.63	4.1	4.7	5.4	6.1
Medium gravel	0.63 – 0.31	3.3	3.7	4.1	4.6
Fine gravel	0.31 – 0.16	2.6	3.0	3.3	3.8
Very fine gravel	0.16 – 0.079	2.2	2.5	2.8	3.1
Very coarse sand	0.079 – 0.039	1.8	2.1	2.4	2.7
Coarse sand	0.039 – 0.020	1.5	1.8	2.1	2.3
Medium sand	0.020 – 0.010	1.2	1.5	1.8	2.0
Fine sand	0.010 – 0.005	0.98	1.3	1.6	1.8
Compact cohesive soils					
Heavy sandy loam		3.3	3.9	4.6	4.9
Light		3.1	3.9	4.6	4.9
Loess soils in the conditions of finished settlement		2.6	3.3	3.9	4.3

Notes:

(1) Bed materials may move if velocities are higher than the nonscour velocities.

(2) Mean depth is calculated by dividing the cross-sectional area of the waterway by the top width of the water surface. If the waterway can be subdivided into a main channel and an overbank area, the mean depths of the channel and the overbank should be calculated separately. For example, if the size of moving material in the main channel is desired, the mean depth of the main channel is calculated by dividing the cross-sectional area of the main channel by the top width of the main channel.

Table 855.2C

Guide for Anticipated Service Life Added to Steel Pipe by Abrasive Resistant Protective Coating (2)

Flow Velocity (ft/s)	Channel Materials	Bituminous Coating (yrs.) (hot-dipped)	Bituminous Coating & Paved Invert (yrs.)	Polymeric Sheet Coating (yrs.)	Polyethylene (CSSRP) (yrs.)
	Non-Abrasive	8	15	*	*
≥ 1 – ≤ 8 (1)	Abrasive	6-0	15-2	30-5	*
> 8 – ≤ 12	Abrasive	0	2-0	5-0	70-35
> 12 – ≤ 15	Abrasive	**	**	**	35-8***
> 12 – ≤ 20	Abrasive & heavy bedloads	****	****	****	****

* Provides adequate abrasion resistance to meet or exceed a 50-year design service life.

** Abrasive resistant protective coatings not recommended, increase steel thickness to 10 gage.

*** Not recommended above 14 fps flow velocity.

**** Contact District Hydraulics Branch. See Table 855.2F.

Notes:

(1) Where there are increased velocities with minor bedload volumes, much higher velocities may be applicable.
(2) Range of additional service life commensurate with flow velocity range.

Table 855.2D

Guide for Anticipated Wear to Metal Pipe by Abrasive Channel Materials

Flow Velocity (ft/s)	Channel Materials	Anticipated Wear (mils/yr)		
		Plain Galvanized	Aluminized Steel (Type 2)	Aluminum**
	Non-Abrasive	0*	0*	0
≥ 1 – ≤ 8	Abrasive	0*	0*	0 – 1.5
> 8 – ≤ 12	Abrasive	0.5 – 1	0.5 – 1	1.5 – 3
> 12 – ≤ 15	Abrasive	1 – 3.5	1 – 3.5	3 – 10.5
> 12 – ≤ 20	Abrasive & Heavy bedloads	2.5 – 10	2.5 – 10	7.5 – 30

* Refer to California Test 643 and Figure 855.3B.

** Refer to Figure 855.3A.

Note:

1 mil = 0.001"

Table 855.2E

Relative Abrasion Resistance Properties of Pipe and Lining Materials*

Material	Relative Wear (dimensionless)
Steel	1
Aluminum	1.5 – 3
PVC	2
Polyester Resin (CIPP)	2.5 – 4
HDPE	4 – 5
Concrete (RCP 4000 – 7000 psi)	75 – 100
Calcium Aluminate (Mortar)	30-40
Calcium Aluminate (Concrete)	20 – 25
Basalt Tile	1
Polyethylene (CSSRP)	1 – 2

* Evaluation of Abrasion Resistance of Pipe and Pipe Lining Materials Final Report FHWA/CA/TL-CA01-0173 (2007).

Table 855.2F

Guide for Minimum Material Thickness of Abrasive Resistant Invert Protection to Achieve 50 Years of Maintenance-Free Service Life

Abrasion Level & Flow Velocity (ft/s)	Channel Materials	Concrete (4) (in)	Steel Pipe & Plate (in)	Aluminum Pipe & Plate (in)	PVC (in)	HDPE (in)	CIPP (in)	Calcium Aluminate Abrasion Resistant Concrete (5) (in)	Mortar (5)	
									Calcium Aluminate (in)	Geopolymer (in)
Level 4 > 8 – ≤ 12	Abrasive	2 – 4	0.052	0.075 – 0.164	0.1	0.125 – 0.25	0.1 – 0.3	(6)	1-2	2-4
Level 5 > 12 – ≤ 15	Abrasive	4 – 13	0.052 – 0.18	(2)	0.1 – 0.35	0.25 – 0.875	0.3 – 0.70	3(6)	2-5	4-13
Level 6 > 12 – ≤ 20	Abrasive & Heavy bedloads	(1)	0.109 – 0.5	(2)	0.25 – 1.0(3)	0.625 – 2.5	0.5 – 2	3 – 5	5-8	(1)

Notes:

(1) For flow velocity > 12 ft/s ≤ 14 ft/s use 9" – 15". For > 14 ft/s use CRSP or other abrasion resistant layer special design with, or in lieu of concrete or geopolymer mortar.

(2) Not recommended without invert protection.

(3) PVC liners not recommended when freezing conditions are often encountered and cobbles and rocks are present.

(4) Values shown based on RCP abrasion test results. See Table 855.2E. Results may differ from concrete specified under 15-6.04 for invert paving which must have a minimum compressive strength of 6,000 psi at 28 days and 1 ½-inch maximum grading.

(5) See Authorized Materials List for Cementitious Pipeliners and Concrete Invert Paving: http://www.dot.ca.gov/hq/esc/approved_products_list/
Standard Mortar (Section 51-1.02F of the Standard Specifications) not recommended for Abrasion Level 4 or higher.

(6) Minimum thickness recommended is 3". Not practical or economically viable for Level 4. Consider calcium aluminate mortar or standard concrete (Section 90 of the Standard Specifications) for lower range of Level 5.

culverts smaller than 30 inches or larger diameters with insignificant abrasive bedload volumes).

Abrasion resistance for any concrete lining is dependent upon the thickness, quality, strength, and hardness of the aggregate and compressive strength of the concrete as well as the velocity of the water flow coupled with abrasive sediment content and acidity. Abrasion resistant concrete or mortar made from calcium aluminate provides much improved abrasion resistance over cementitious concrete and should be considered as a viable countermeasure in extremely abrasive conditions (i.e, velocity greater than 15 feet per second with heavy bedload). See Table 855.2F.

Plastic materials typically exhibit good abrasion resistance but service life is constrained by the manufactured thickness of typical pipe profiles. Both PVC and HDPE corrugated pipe are limited for their use in moderate and heavy bedload abrasion conditions by the combined manufactured inner liner and corrugated wall thicknesses. For culvert rehabilitation, PVC and HDPE pipe slip lining products (e.g. solid wall HDPE) are viable options for applications in moderate and heavy bedload abrasion conditions (see Table 855.2A).

Table 855.2A can be used as a “preliminary estimator” of abrasion potential for material selection to achieve the required service life, however, it incorporates only three of the primary abrasion factors; bedload volume, bedload type and flow velocity and the general assumption is the materials are angular, hard and abrasive. As discussed above, the other factors that are not used in the table should also be carefully considered. For example, under similar hydraulic conditions, heavy volumes of hard, angular sand may be more abrasive than small volumes of relatively soft, large or rounded rocks. Furthermore, two sites with similar site characteristics, but different hydrologic characteristics, i.e., volume, duration and frequency of stream flow in the culvert, will likely also have different abrasion levels. Table 855.2B can be used as a guide with Table 855.2A to determine the maximum size of material that can be moved through a pipe. Field observations of channel bed material both upstream and downstream from the pipe are extremely important for estimating the size range of transportable material in the channel.

855.3 Corrosion

Corrosion is the destructive attack on a pipe by a chemical reaction with the materials surrounding the pipe. Corrosion problems can occur when metal pipes are used in locations where the surrounding materials have excess acidity or alkalinity. The relative acidity of a substance is often expressed by its pH value. The pH scale ranges from 1 to 14, with 1 representing extreme acidity, and 14 representing extreme alkalinity, and 7 representing a neutral substance. The closer the pH value is to 7, the less potential the substance has for causing corrosion.

Corrosion is an electrolytic process and requires an electrolyte (generally moisture) and oxygen to proceed. As a result, it has the greatest potential for causing damage in soils that have a relative high ability to pass electric current. The ability of a soil to convey current is expressed as its resistivity in ohm-cm, and a soil with a low resistivity has a greater ability to conduct electricity. Very dry areas (e.g., desert environments) have a limited availability of electrolyte, and totally and continuously submerged pipes have limited oxygen availability. These extreme conditions (among others) are not well represented by AltPipe, and some adjustment in the estimated service life for pipes in these conditions should be made. See Index 857.2

Corrosion can also be caused by excessive acidity in the water conveyed by the pipe. Water pH can vary considerably between watersheds and seasons.

Because failure can occur at any point along the length of the pipe (e.g. tidal zones), the designer must look at the conditions and how they may vary along the pipe length - and select for input into AltPipe those conditions that represent the most severe situation along the length.

AltPipe operates based on some fairly basic assumptions for corrosion and minimum resistivity that are part of California Test 643. Altpipe will list all viable alternatives for achieving design service life. Where enhanced soilside corrosion protection is needed, aluminum or aluminized pipe (if within acceptable pH/min. resistivity ranges), bituminous coatings or polymeric sheet coating should be considered.

Aluminum, and the aluminum coating provided by Aluminized Steel (Type 2) pipe, corrodes differently than steel and will provide adequate durability to meet the 50-year service life criterion within the acceptable pH range of 5.5-8.5 and minimum resistivity greater than 1500 ohm-cm without need for specifying a thicker gauge or additional coating, whereas under the same range galvanized steel may need a protective coating or an increase in thickness to provide a 50-year maintenance-free service life (with respect to corrosion). Figure 855.3A should be used to determine the limitations on the use of corrugated aluminum pipe for various levels of pH and minimum resistivity. The minimum thickness (0.060 inch) of aluminum pipe obtained from the chart only satisfies corrosion requirements. Overfill requirements for minimum metal thickness must also be satisfied. The metal thickness of corrugated aluminum pipe should satisfy both requirements.

Figure 855.3A should be used to determine the minimum thickness and limitation on the use of corrugated steel and spiral rib pipe for various levels of pH and minimum resistivity. For example, given a soil environment with pH and minimum resistivity levels of 6.5 and 15,000 ohm-cm, respectively, the minimum thicknesses for the various metal pipes are: 1) 0.109 inch (12 gage) galvanized steel, 2) 0.064 inch (16 gage) aluminized steel (type 2) and 3) 0.060 inch (16 gage) aluminum. The minimum thickness of metal pipe obtained from the figure only satisfies corrosion requirements. Overfill requirements for minimum metal thickness must also be satisfied. The metal thickness of corrugated pipe and steel spiral rib pipe that satisfies both requirements should be used.

Figure 855.3B, “Chart for Estimating Years to Perforation of Steel Culverts,” is part of a Standard California Department of Transportation Test Method derived from highway culvert investigations. This chart alone is not used for determining service life because it does not consider the effects of abrasion or overfill; it is for estimating the years to the first corrosion perforation of the wall or invert of the CSP.

855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates

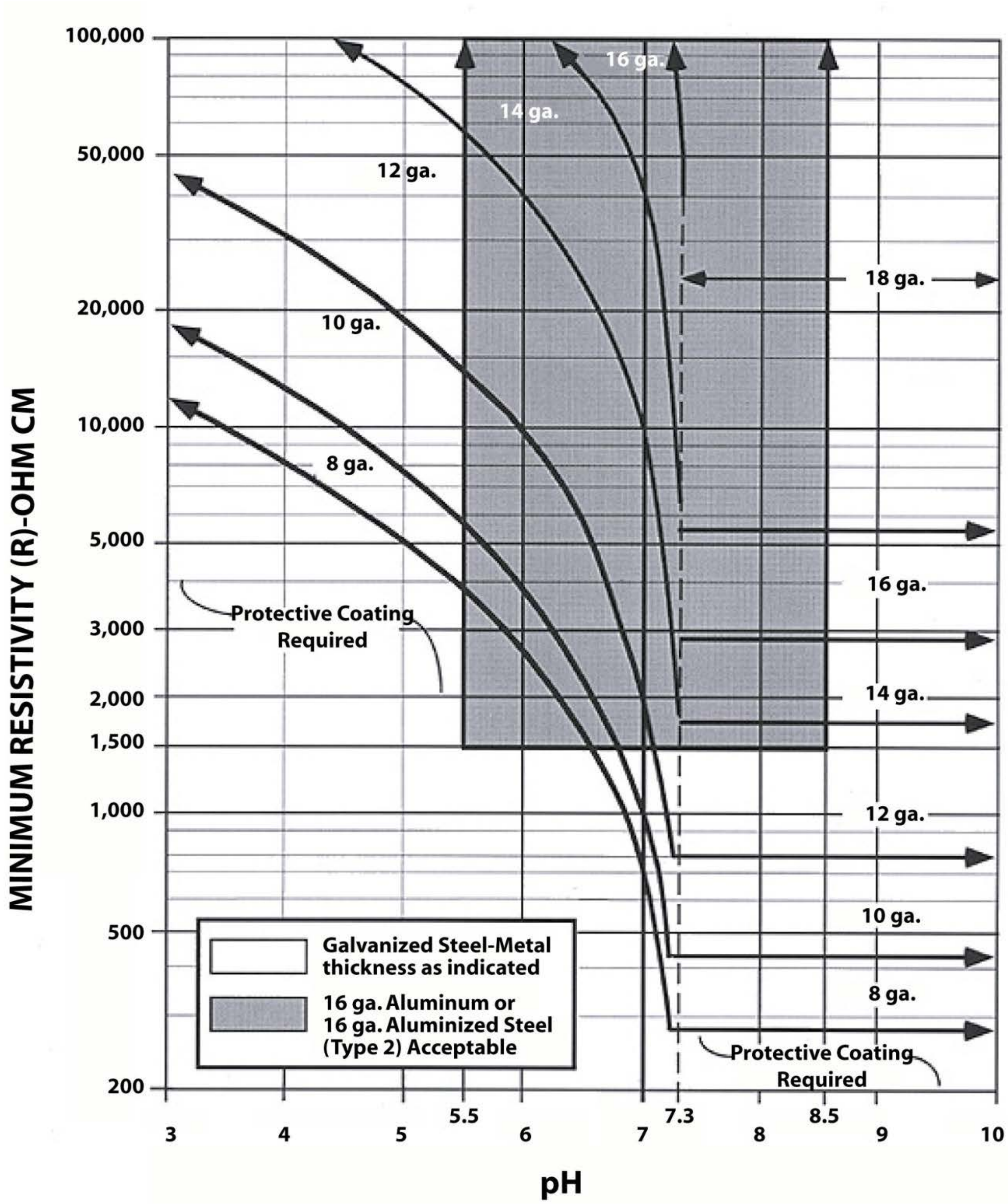
Table 855.4A indicates the limitation on the use of concrete by acidity of soil and water. Table 855.4A is also a guide for designating cementitious material restrictions and water content restrictions for various ranges of sulfate concentrations in soil and water for all cast in place and precast construction of drainage structures.

For pH ranging between 7.0 and 3.0 and for sulfate concentrations between 1500 and 15,000 ppm, concrete mix designs conforming to the recommendations given in Table 855.4A should be followed. Higher sulfate concentrations or lower pH values may preclude the use of concrete or would require the designer to develop and specify the application of a complete physical barrier. Reinforcing steel can be expected to respond to corrosive environments similarly to the steel in CSP.

Table 855.4B provides a guide for minimum concrete cover requirements for various ranges of chloride concentrations in soil and water for all precast and cast in place construction of drainage structures.

(1) *RCP*. In relatively severe acidic, chloride or sulfate environments (either in the soil or water) as identified in the project Materials Report, the means for offsetting the effects of the corrosive elements is to either increase the cover over the reinforcing steel, increase the cementitious material content, or reduce the water/ cementitious material ratio. The identified constituent concentration levels should be entered into AltPipe to verify what combinations of increased cover (in 1/4-inch intervals from 1 inch to a maximum of 1-1/2 inches), increased cementitious material content (in increments of 47 pounds from 470 pounds to a maximum of 564 pounds), will provide the necessary service life (typically 50 years). Per an agreement with Industry, the water to cementitious material ratio is set at 0.40. AltPipe is specifically programmed to provide RCP mix and cover designs that are compatible with industry practice, and are based on their agreements with Caltrans. For corrosive condition installations such as low pH (<4.5),

Figure 855.3A
Minimum Thickness of Metal Pipe
for 50-Year Maintenance-Free Service Life (2)



Notes:

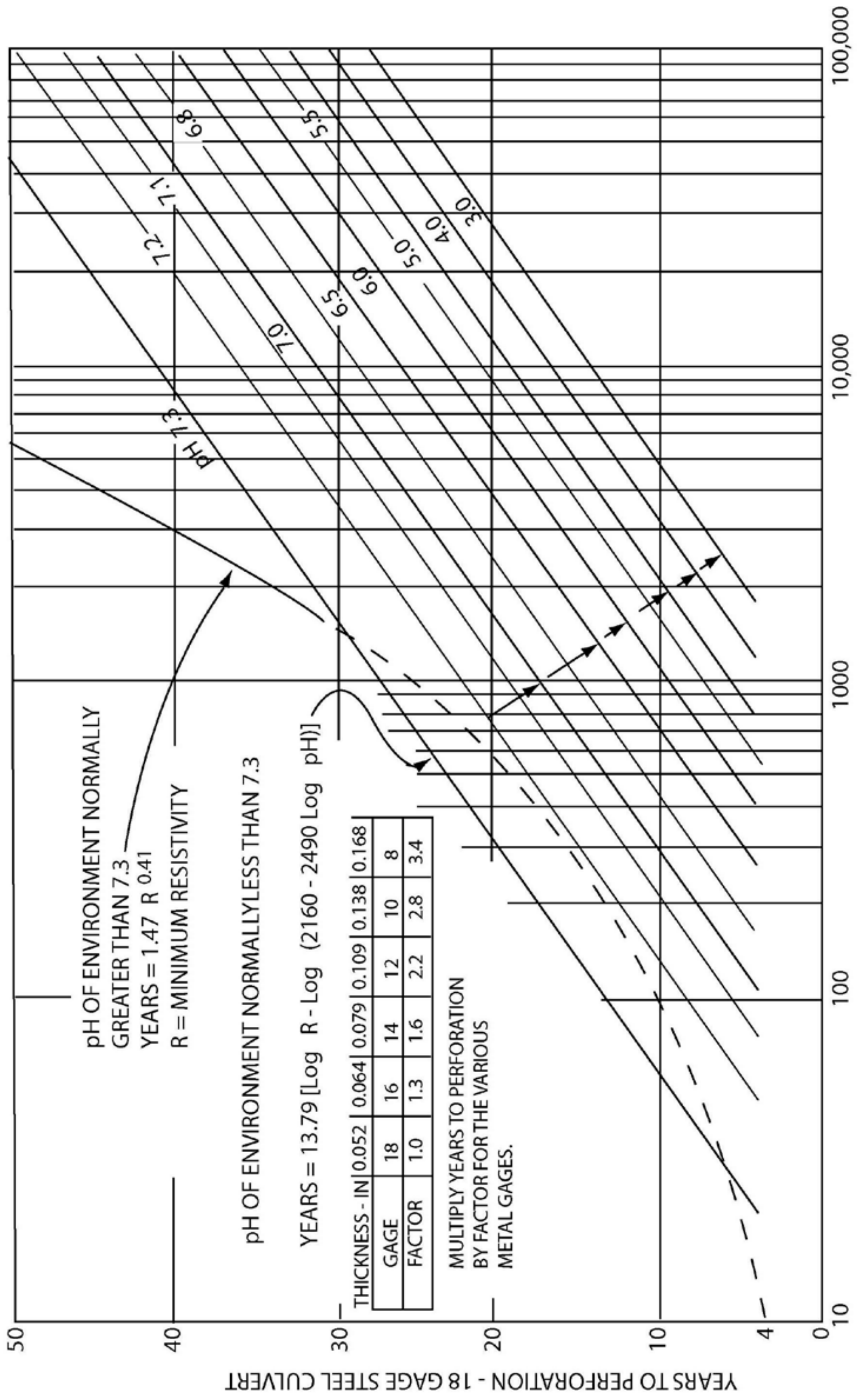
(1) For pH and aluminum resistivity levels not shown refer to Fig. 855.3B steel pipes. (California Test 643)

(2) Service life estimate are for various corrosive conditions only.

(3) Refer to Index 852.3(2) and 852.4(2) for appropriate selection of metal thickness and protection coating to achieve service life requirements.

Figure 855.3B

Chart for Estimating Years to Perforation of Steel Culverts



Chlorides (>2,000 ppm) or Sulfates (> 2,000 ppm), the following service life (SL) equation provides the basis for RCP design in AltPipe:

$$SL = 10^3 \times 1.107^{Cc} \times Cc^{0.717} \times Dc^{1.22} \times (K+1)^{-0.37} \times W^{-0.631} - 4.22 \times 10^{10} \times pH^{-14.1} - 2.94 \times 10^{-3} \times S + 4.41$$

Where: S = Environmental sulfate content in ppm.

Cc = Sacks of cement (94 lbs each) per cubic yard of concrete.

Dc = Concrete cover in inches.

K = Environmental chloride concentration in ppm.

W = Water by volume as percentage of total mix.

pH = The measure of relative acidity or alkalinity of the soil or water. See Index 855.3.

Where the measured concentration of chlorides exceeds 2000 ppm for RCP that is placed in brackish or marine environments and where the high tide line is below the crown of the invert, the AltPipe input for chloride concentration will default to 25,000 ppm.

Contact the District Materials unit or the Corrosion Technology Branch in DES for design recommendations when in extremely corrosive conditions. Non-Reinforced concrete pipe is not affected by chlorides or stray currents and may be used in lieu of RCP with additional concrete cover and/or protective coatings for sizes 36" in diameter and smaller. See Index 852.1(4) and Table 855.4A. Where conditions occur that RCP designs as produced by AltPipe will not work, the Office of State Highway Drainage Design within the Division of Design should be contacted.

855.5 Material Susceptibility to Fire

Fire can occur almost anywhere on the highway system. Common causes include forest, brush or grass fires that either enter the right-of-way or begin within it. Less common causes include spills of flammable liquids that ignite or vandalism. Storm drains, which are completely buried would typically be impacted by spills or vandalism. Because these are such low probability events, prohibitions on material placement for storm drains are not typically warranted.

Cross culverts and exposed overside drains are the placement types most subject to burning or melting and designers should consider either limiting the alternative pipe listing to non-flammable pipe materials or providing a non-flammable end treatment to provide some level of protection.

Plastic pipe and pipes with coatings (typically of bituminous or plastic materials) are the most susceptible to damage from fire. Of the plastic pipe types which are allowed, PVC will self extinguish if the source of the fire is eliminated (i.e., if the grass or brush is consumed or removed) while HDPE can continue to burn as long as an adequate oxygen supply is present. Based on testing performed by Florida DOT, this rate of burning is fairly slow, and often self extinguished if the airflow was inhibited (i.e., pipe not aligned with prevailing wind or ends sheltered from air flow).

Due to the potential for fire damage, plastic pipe is not recommended for overside drain locations where there is high fire potential (large amounts of brush or grass or areas with a history of fire) and where the overside drain is placed or anchored on top of the slope.

Where similar high fire potential conditions exist for cross culverts, the designer may consider limiting the allowable pipe materials indicated on the alternative pipe listing to non-flammable material types, use concrete endwalls that eliminate exposure of the pipe ends, or require that the end of flammable pipe types be replaced with a length of non-flammable pipe material.

Topic 856 - Height of Fill

An essential aspect of pipe selection is the height of fill/cover over the pipe. This cover dissipates live loads from traffic, both during construction and after the facility is open to the public.

856.1 Construction Loads

See Standard Plan D88 for table of minimum cover for construction loads.

Table 855.4A

Guide for the Protection of Cast-In-Place and Precast Reinforced and Unreinforced Concrete Structures[5] Against Acid and Sulfate Exposure Conditions[1],[2]

Soil or Water pH	Sulfate Concentration of Soil or Water (ppm)	Cementitious Material Requirements [3]	Water Content Restrictions
7.1 to 14	0 to 1,499	Standard Specifications Section 90	No Restrictions
5.6 to 7.0	1,500 to1,999	Standard Specifications Section 90	Maximum water-to-cementitious material ratio of 0.45
3 to 5.5[4]	2,000 to 15,000[4]	675 lb/cy minimum: Type II or Type V portland cement and required supplementary cementitious materials per Standard Specification 90-1.02H	Maximum water-to-cementitious material ratio of 0.40

Notes:

(1) Recommendations shown in the table for the cementitious material requirements and water content restrictions should be used if the pH and/or the sulfate conditions in Column 1 and/or Column 2 exists. Sulfate testing is not required if the minimum resistivity is greater than 1,000 ohm-cm.

(2) The table lists soil/water pH and sulfate concentration in increasing level of severity starting from the top of the table. If the soil/water pH and the sulfate concentration are at different levels of severity, the recommendation for the more severe level will apply. For example, a soil with a pH of 4.0, but with a sulfate concentration of only 1,600 ppm would require a minimum of 675 lb/cy of cementitious material. The maximum water-to-cementitious material ratio would be 0.40.

(3) Cementitious material shall conform to the provisions in Section 90 of the Standard Specifications.

(4) Additional mitigation measures will be needed for conditions where the pH is less than 3 and/or the sulfate concentration exceeds 15,000 ppm. Mitigation measures may include additional concrete cover and/or protective coatings. For additional assistance, contact the Corrosion Technology Branch of Materials Engineering and Testing Services (METS) at 5900 Folsom Boulevard Sacramento, CA. 95819.

(5) Does not include RCP.

Table 855.4B

Guide for Minimum Cover Requirements for Cast-In-Place and Precast Reinforced Concrete Structures[3] for 50-Year Design Life in Chloride Environments

Chloride Concentration (ppm)			
500 to 2000	2001 to 5000	5001 to 10000	10000 +
1.5 in.[1]	2.5 in.[1]	3 in.[1]	4 in.[1]
1.5 in.[2]	1.5 in.[2]	2 in.[2]	3 in.[2]

Notes:

(1) Supplementary cementitious materials are required. Typical minimum requirement consists of 675#/cy minimum cementitious material with 75% by weight of Type II or Type V portland cement and 25% by weight of either fly ash or natural pozzolan. A maximum w/cm ratio of 0.40 is specified. Fly ash or natural pozzolan may have a CaO content of up to 10%. Section 90-1.02B(3) of the Standard Specifications provides requirements.

(2) Additional supplementary cementitious materials per the requirements of Section 90-1.02B(3) of the Standard Specifications are required in order to achieve the listed reduction in concrete cover.

(3) Does not include RCP.

856.2 Concrete Pipe, Box and Arch Culverts

(1) *Reinforced Concrete Pipe.* See Standard Plan A62D and A62DA for the maximum height of overfill for reinforced concrete pipe, up to and including 120-inch diameter (or reinforced oval pipe and reinforced concrete pipe arch with equivalent cross-sectional area), using the backfill method or type shown. For oval shaped reinforced concrete pipe fill heights, see Standard Plan A62D and Indirect Design D-Load (Marsten/Spangler Method). Allowable cover for oval shaped reinforced concrete pipe is determined by using Method 2 (Note 8). See Standard Plan D79 and D79A for pre-cast reinforced concrete pipe Direct Design Method (pertains to circular pipe only).

The designer should be aware of the premises on which the tables on Standard Plan A62D, A62DA, D79 and D79A are computed as well as their limitations. The cover presupposes:

- That the bedding and backfill satisfy the terms of the Standard Specifications, the conditions of cover and pipe size required by the plans, and take into account the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert equal in magnitude to that of the adjoining material outside the trench.
- Subexcavation and backfill as required by the Standard Specifications where unyielding foundation material is encountered.

If the height of overfill exceeds the tabular values on Standard Plan A62D and A62DA a special design is required; see Index 829.2.

(1) *Concrete Box and Arch Culverts.* Single and multiple span reinforced concrete box culverts are completely detailed in the Standard Plans. For cast-in-place construction, strength classifications are shown for 10 feet and 20 feet overfills. See Standard Plan numbers D80, D81 and D82. Pre-cast reinforced concrete box culverts require a minimum of 1 foot overfill and limit fill height to 12 feet maximum. See Standard Plans D83A, D83B and A62G. For fill height design criteria for CIP Bottomless 3-sided rigid frame culverts see XS-Sheets 17-050-1, 2, 3, 4 and 5. Cast-in-place reinforced concrete arch culverts are no longer economically feasible structures and last appeared in the 1997 Standard Plans. Questions regarding fill height for concrete arch culverts or extensions should be directed to the Underground Structures Branch of DES - Structures Design.

856.3 Metal Pipe and Structural Plate Pipe

Basic Premise - To properly use the fill height design tables, the designer should be aware of the premises on which the tables are based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications and Standard Plan A62F, the conditions of cover, and pipe size required by the plans and the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.

Limitations - In using the tables, the following restrictions must be kept in mind:

- The values given for each size of pipe constitute the maximum height of overfill or cover over the pipe for the thickness of metal and kind of corrugation.
- The thickness shown is the structural minimum. Where abrasive conditions are anticipated, additional metal thickness or invert treatments as stated under Index 852.4(5) and Index 852.6(2)(c) should be provided when required to fulfill the design service life requirements of Topic 855.
- Where needed, adequate provisions for corrosion resistance must be made to achieve the required design service life called for in the references mentioned herein.
- Table 856.3D shows the limit of heights of cover for corrugated steel pipe arches based on the supporting soil sustaining a factored bearing pressure varying between 3.38 tons per square feet to 3.55 tons per square feet. Table 856.3J shows similar values for corrugated aluminum pipe arches.

- The values given for each size of structural plate pipe or arch constitute the maximum height of overfill or cover over the pipe or arch for the thickness of metal and kind of corrugation.
- Tables 856.3N & P show the limit of heights of cover for structural plate arches based on the supporting soil sustaining a factored bearing pressure of 6 tons per square foot at the corners.

Special Designs.

- If the height of overfill exceeds the tabular values, or if the foundation investigation reveals that the supporting soil will not develop the bearing pressure on which the overfill heights for pipe arches are based, a special design prepared by DES - Structures Design is required. See index 829.2.
- Non-standard pipe diameters and arch sizes are available. Loading capacity of special designs needs to be verified with the Underground Structures Branch of DES - Structures Design.
- Aluminum pipe fill height tables are based on use of H-32 temper aluminum. If use of aluminum is necessary and greater structural capacity is required, H-34 temper can be specified. Contact Underground Structures branch of DES-Structures Design for calculation of allowable fill height.

(1) *Corrugated Steel Pipe and Pipe Arches, Steel Spiral Rib Pipe, Structural Steel Plate Pipe and Structural Steel Plate Pipe Arches*. The allowable overfill heights for corrugated steel pipe and pipe arches for the various diameters or arch sizes and metal thickness are shown on Tables 856.3A, B, C & D. For steel spiral rib pipe, overfill heights are shown on Tables 856.3E, F, G & H. Table 856.3G gives the allowable overfill height for composite steel spiral rib pipe.

For structural steel plate pipe and structural steel plate pipe arches, overfill heights are shown on Tables 856.3M & N. For maximum height of fill over structural steel plate vehicular undercrossings, see Standard Plan B14-1.

(2) *Corrugated Aluminum Pipe and Pipe Arches, Aluminum Spiral Rib Pipe and Structural Aluminum Plate Pipe and Structural Aluminum Plate Pipe Arches.* The allowable overfill heights for corrugated aluminum pipe and pipe arches for various diameters and metal thickness are shown on Tables 856.3H, I & J. For aluminum spiral rib pipe, overfill heights are shown on Tables 856.3K & L.

For structural aluminum plate pipe and structural aluminum plate pipe arches, overfill heights are shown on Tables 856.3O, & P.

856.4 Plastic Pipe

The allowable overfill heights for plastic pipe for various diameters are shown in Tables 856.4 and 856.5. To properly use the plastic pipe height of fill table, the designer should be aware of the basic premises on which the table is based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications and Standard Plan A62F, the conditions of cover, and pipe size required by the plans and the essentials of Index 829.2.
- That corrugated high density polyethylene (HDPE) pipe that is greater than 48" in size shall be backfilled with cementitious (slurry cement, CLSM or concrete) backfill.
- That where cementitious or flowable backfill is used for structural backfill, the backfill shall be placed to a level not less than 12 inches above the crown of the pipe.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.
- That the average water table elevation is at or below the pipe springline.
- Corrugated HDPE pipe, Type C is recommended for placement only outside the roadbed where vehicular loading is unlikely (e.g., overside drains, medians) unless cementitious backfill is specified.

856.5 Minimum Height of Cover

Table 856.5 gives the minimum thickness of cover required for design purposes over pipes and pipe arches. For construction purposes, a minimum cover of 6 inches greater than the roadway structural section is desirable for all types of pipe.

Table 856.3A

Corrugated Steel Pipe
Helical Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)					
	Metal Thickness (in)					
	0.052 (18 ga.)	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)	0.168 (8 ga.)
	2⅔" x ½" Corrugations					
12-15	118	148	177	--	--	--
18	99	124	148	207	--	--
21	85	106	132	177	--	--
24	74	93	116	155	200	245
30	59	74	93	130	160	195
36	49	62	77	108	139	163
42	42	53	66	93	119	139
48	--	46	58	81	104	128
54	--	--	51	72	93	113
60	--	--	--	65	83	102
66	--	--	--	--	76	93
72	--	--	--	--	70	85
78	--	--	--	--	--	75
84	--	--	--	--	--	65
	3" x 1" Corrugations					
48	--	53	67	93	120	147
54	--	47	59	83	107	131
60	--	42	53	75	96	118
66	--	39	48	68	87	107
72	--	35	44	62	80	98
78	--	33	41	57	74	91
84	--	30	38	53	69	84
90	--	28	35	50	64	78
96	--	--	33	47	60	74
102	--	--	31	44	56	69
108	--	--	--	41	53	65
114	--	--	--	39	50	62
120	--	--	--	37	48	59

Table 856.3B

Corrugated Steel Pipe
Helical Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)			
	Metal Thickness (in)			
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)
	5" x 1" Corrugations			
48	47	59	83	--
54	42	53	74	95
60	38	47	66	86
66	34	43	60	78
72	31	39	55	71
78	29	36	51	66
84	27	34	47	61
90	25	31	44	57
96	--	29	41	53
102	--	28	39	50
108	--	--	37	47
114	--	--	35	45
120	--	--	33	43

Table 856.3C

Corrugated Steel Pipe
2⅔" x ½" Annular Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)				
	Metal Thickness (in)				
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)	0.168 (8 ga.)
18	54	--	--	--	--
21	46	--	--	--	--
24	40	44	--	--	--
30	32	35	--	--	--
36	27	29	38	--	--
42	30	41	65	68	--
48	26	36	57	59	62
54	--	32	50	53	55
60	--	--	45	47	50
66	--	--	--	43	45
72	--	--	--	39	41
78	--	--	--	--	38
84	--	--	--	--	35

Table 856.3D

Corrugated Steel Pipe Arches
2⅔" x ½" Helical or Annular Corrugations

			MAXIMUM HEIGHT OF COVER (ft)			
			Metal Thickness (in)			
Span-Rise (in)	Factored Bearing Demand (tons/ft^2)	Minimum Corner Radius (in)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)	0.168 (8 ga.)
21 x 15	3.50	4 1/8	10	--	--	--
24 x 18	3.38	4 7/8	10	--	--	--
28 x 20	3.49	5 1/2	10	--	--	--
35 x 24	3.49	6 7/8	10	--	--	--
42 x 29	3.49	8 1/4	10	--	--	--
49 x 33	3.49	9 5/8	10	--	--	--
57 x 38	3.55	11	--	10	--	--
64 x 43	3.54	12 3/8	--	10	--	--
71 x 47	3.54	13 3/4	--	--	10	--
77 x 52	3.49	15 1/8	--	--	--	10
83 x 57	3.45	16 1/2	--	--	--	10

Note:
(1) Cover limited by corner soil bearing pressure as shown.

Table 856.3E

Steel Spiral Rib Pipe
¾" x 1" Ribs at 11½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)		
	Metal Thickness (in)		
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)
24	44	62	105
30	36	50	84
36	30	42	70
42	25	36	60
48	22	31	53
54	20	28	47
60	--	25	42
66	--	22	38
72	--	21	35
78	--	--	32
84	--	--	30
90	--	--	28
96	--	--	--

Table 856.3F

Steel Spiral Rib Pipe
¾" x 1" Ribs at 8½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)		
	Metal Thickness (in)		
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)
24	59	83	137
30	48	66	110
36	40	55	92
42	34	47	78
48	30	41	69
54	26	37	61
60	24	33	55
66	21	30	50
72	20	27	46
78	--	25	42
84	--	23	39
90	--	--	36
96	--	--	34
102	--	--	32
108	--	--	30
114	--	--	--

Table 856.3G

Steel Spiral Rib Pipe
¾” x ¾" Ribs at 7½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)			
	Metal Thickness (in)			
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)
24	61	85	141	205
30	49	68	113	164
36	40	57	94	137
42	35	48	81	117
48	30	42	71	103
54	27	38	63	91
60	--	34	57	82
66	--	31	51	75
72	--	--	47	68
78	--	--	43	63
84	--	--	40	59
90	--	--	--	55

Table 856.3H
Corrugated Aluminum Pipe
Annular Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft) — Metal Thickness (in)				
	0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)	0.135 (10 ga.)	0.164 (8 ga.)
	2⅔" x ½" Corrugations				
12	43	43	--	--	--
15	35	34	60	--	--
18	29	29	50	--	--
21	25	25	43	--	--
24	21	21	37	39	--
30	--	17	30	31	--
36	--	14	25	26	--
42	--	--	43	45	--
48	--	--	38	40	41
54	--	--	34	35	36
60	--	--	--	32	33
66	--	--	--	--	30
72	--	--	--	--	27
	3" x 1" Corrugations				
30	32	40	54	81	--
36	26	33	45	68	88
42	23	28	39	58	75
48	20	25	34	51	66
54	17	22	30	45	59
60	16	20	27	41	53
66	14	18	24	37	48
72	13	16	22	34	44
78	--	15	21	31	40
84	--	--	19	29	38
90	--	--	18	27	35
96	--	--	17	25	33
102	--	--	--	24	31
108	--	--	--	22	29
114	--	--	--	--	28
120	--	--	--	--	26

Table 856.3I
Corrugated Aluminum Pipe
Helical Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)				
	Metal Thickness (in)				
	0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)	0.135 (10 ga.)	0.164 (8 ga.)
	2⅔" x ½" Corrugations				
12	112	140	--	--	--
15	90	112	156	--	--
18	75	93	130	--	--
21	64	80	112	--	--
24	56	70	98	126	--
30	--	56	78	101	--
36	--	47	65	84	--
42	--	--	56	72	--
48	--	--	49	63	77
54	--	--	43	56	68
60	--	--	--	46	58
66	--	--	--	--	47
72	--	--	--	--	37
	3" x 1" Corrugations				
30	51	65	90	121	--
36	43	54	75	101	118
42	37	46	64	86	102
48	32	40	56	76	89
54	28	36	50	67	79
60	26	32	45	60	71
66	23	29	41	55	65
72	21	27	37	50	59
78	--	25	35	46	55
84	--	--	32	43	51
90	--	--	30	40	47
96	--	--	28	38	44
102	--	--	--	35	42
108	--	--	--	33	39
114	--	--	--	--	36
120	--	--	--	--	32

Table 856.3J

Corrugated Aluminum Pipe Arches
2⅔" x ½" Helical or Annular Corrugations

Span-Rise (in)	Factored Bearing Demand (tons/ft^2)	Minimum Corner Radius (in)	MAXIMUM HEIGHT OF COVER (ft) Metal Thickness (in) 0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)	0.135 (10 ga.)	0.164 (8 ga.)
17 x 13	3.34	3 1/2	10	--	--	--	--
21 x 15	3.49	4 1/8	10	--	--	--	--
24 x 18	3.38	4 7/8	10	--	--	--	--
28 x 20	3.49	5 1/2	--	10	--	--	--
35 x 24	3.49	6 7/8	--	10	--	--	--
42 x 29	3.49	8 1/4	--	--	10	--	--
49 x 33	3.49	9 5/8	--	--	10	--	--
57 x 38	3.55	11	--	--	--	10	--
64 x 43	3.54	12 3/8	--	--	--	10	--
71 x 47	3.54	13 3/4	--	--	--	--	10

Note:
(1) Cover is limited by corner soil bearing pressure as shown.

Table 856.3K

Aluminum Spiral Rib Pipe
¾" x 1” Ribs at 11½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)		
	Metal Thickness (in)		
	0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)
24	22	31	50
30	18	24	40
36	15	20	33
42	--	17	29
48	--	--	25
54	--	--	22
60	--	--	20
66	--	--	--
72	--	--	--

Table 856.3L

Aluminum Spiral Rib Pipe
¾" x ¾" Ribs at 7½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)		
	Metal Thickness (in)		
	0.60 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)
24	30	41	66
30	24	33	53
36	20	27	44
42	--	23	38
48	--	--	33
54	--	--	29
60	--	--	26
66	--	--	--
72	--	--	--

Table 856.3M
Structural Steel Plate Pipe
6" x 2" Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)							
	Metal Thickness (in)							
	0.110 (12 ga.)	0.140 (10 ga.)	0.170 (8 ga.)	0.218 (5 ga.)	0.249 (3 ga.)	0.280 (1 ga.)	0.318 (0 ga.)	0.380 (000 ga.)
60	42	60	79	105	128	140	223	268
66	38	55	71	99	116	127	203	243
72	35	50	65	91	107	116	186	223
77	32	47	61	85	100	109	174	209
84	30	43	56	78	92	100	160	192
90	28	40	52	72	85	93	149	179
96	26	37	49	68	80	87	140	168
102	24	35	46	64	75	82	132	158
108	23	33	44	60	71	78	124	149
114	22	31	41	57	67	74	118	141
120	21	30	39	54	64	70	112	134
126	20	28	37	52	61	67	107	128
132	19	27	36	49	58	63	102	122
138	18	26	34	47	56	61	91	117
144	17	25	33	45	53	58	93	112
150	16	24	31	43	51	56	89	108
156	16	23	30	42	49	54	86	103
162	15	22	29	40	47	52	83	100
168	15	21	28	39	46	50	80	96
174	14	20	27	37	44	48	77	93
180	14	20	26	36	43	46	75	90
186	13	19	25	35	41	45	72	87
192	--	18	24	34	40	44	70	84
198	--	18	24	33	39	42	68	81
204	--	17	23	32	38	41	66	79
210	--	17	22	31	36	40	64	77
216	--	--	22	30	35	39	62	75
222	--	--	21	29	34	38	60	73
228	--	--	20	28	34	37	59	71
234	--	--	20	28	33	36	57	69
240	--	--	--	27	32	35	56	67
246	--	--	--	26	31	34	54	65
252	--	--	--	26	30	33	53	64

Table 856.3N

Structural Steel Plate Pipe Arches
6" x 2" Corrugations

MAXIMUM HEIGHT OF COVER (ft)			
Span	Rise	Factored Corner Soil Bearing – 6 tons/ft^2	
		Metal Thickness (in)	
		0.110 (12 ga.)	0.140 (10 ga.)
		18" Corner Radius	
6'-1"	4'-7"	21	--
7'-0"	5'-1"	18	--
7'-11"	5'-7"	16	--
8'-10"	6'-1"	14	--
9'-9"	6'-7"	13	--
10'-11"	7'-1"	12	--
		31" Corner Radius	
13'-3"	9'-4"	17	--
14'-2"	9'-10"	16	--
15'-4"	10'-4"	13	--
16'-3"	10'-10"	12	--
17'-2"	11'-4"	12	--
18'-1"	11'-10"	11	--
19'-3"	12'-4"	--	10
19'-11"	12'-10"	--	10
20'-7"	13'-2"	--	10

NOTES:
(1) For intermediate sizes, the depth of cover may be interpolated.
(2) The 31-inch corner radius arch should be specified when conditions will permit it use.

Table 856.3O

Structural Aluminum Plate Pipe 9" x 2½" Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)						
	Metal Thickness (in)						
	0.100	0.125	0.150	0.175	0.200	0.225	0.250
60	27	40	52	62	71	81	90
66	24	36	48	56	65	73	82
72	22	33	44	51	59	67	75
77	21	31	41	48	55	63	70
84	19	28	37	44	51	58	64
90	18	26	35	41	47	54	60
96	17	25	33	38	44	50	56
102	16	23	31	36	42	47	53
108	15	22	29	34	39	45	50
114	14	21	27	32	37	42	47
120	13	20	26	31	35	40	45
126	13	19	25	29	34	38	43
132	12	18	24	28	32	36	41
138	11	17	23	27	31	35	39
144	--	16	22	25	29	33	37
150	--	16	21	24	28	32	36
156	--	15	20	23	27	31	35
162	--	--	19	23	26	30	33
168	--	--	18	22	25	29	32
174	--	--	18	21	24	28	31
180	--	--	--	20	23	27	30
186	--	--	--	20	23	26	29
192	--	--	--	--	22	25	28
198	--	--	--	--	21	24	27
204	--	--	--	--	--	23	26
210	--	--	--	--	--	23	26
216	--	--	--	--	--	22	25
222	--	--	--	--	--	--	24
228	--	--	--	--	--	--	23

Table 856.3P

Structural Aluminum Plate Pipe Arches
9" x 2½" Corrugations

		MAXIMUM HEIGHT OF COVER (ft)					
		Factored Corner Soil Bearing – 6 tons/ft^2					
		Metal Thickness (in)					
Span	Rise	0.100	0.125	0.150	0.175	0.200	0.225
6'-7"	5'-8"	20	--	--	--	--	--
7'-9"	6'-0"	17	--	--	--	--	--
8'-10"	6'-4"	15	--	--	--	--	--
9'-11"	6'-8"	13	--	--	--	--	--
10'-3"	6'-9"	13	19	--	--	--	--
11'-1"	7'-0"	12	18	20	--	--	--
12'-3"	7'-3"	11	16	18	--	--	--
12'-11"	7'-6"	10	15	17	--	--	--
13'-1"	8'-2"	10	15	17	--	--	--
13'-11"	8'-5"	9	14	16	--	--	--
14'-0"	8'-7"	9	14	16	--	--	--
14'-8"	9'-8"	--	13	15	--	--	--
15'-7"	10'-2"	--	12	13	--	--	--
16'-1"	10'-4"	--	12	13	--	--	--
16'-9"	10'-8"	--	--	12	--	--	--
17'-9"	11'-2"	--	--	--	11	--	--
18'-8"	11'-8"	--	--	--	11	--	--
19'-10"	12'-1"	--	--	--	--	10	--
20'-10"	12'-7"	--	--	--	--	--	9
21'-6"	12'-11"	--	--	--	--	--	9

Note:
(1) 31 inch Corner Radius

Where cover heights above culverts are less than the values shown in Table 856.5, stress reducing slab details available from the Headquarters Design drainage detail library using the following web address may be used: http://onramp.dot.ca.gov/hq/design/drainage/library.php

Where cover heights are less than the values shown in the stress reducing slab details, contact Office of State Highway Drainage Design or the Underground Structures Branch of DES - Structures Design.

Topic 857 - Alternate Materials

857.1 Basic Policy

When two or more materials meet the design service life, and structural and hydraulic requirements, the plans and specifications must provide for alternative pipes, pipe arches, overside drains, and underdrains to allow for optional selection by the contractor. See Index 114.3 (2).

(1) *Allowable Alternatives*. A table of allowable alternative materials for culverts, drainage systems, overside drains, and subsurface drains is included as Table 857.2. This table also identifies the various joint types described in Index 854.1(1) that should be used for the different types of installations.

(2) *Design Service Life*. Each pipe type selected as an alternative must have the appropriate protection as outlined in Topic 852 to assure that it will meet the design service life requirements specified in Topic 855. The maximum height of cover must be in accordance with the tables included in Topic 856.

(3) *Selection of a Specific Material Type*. In the cases listed below, the selection of a specific culvert material must be supported by a complete analysis based on the foregoing factors. All pertinent documentation should be placed on file in the District.

- Where satisfactory performance for a life expectancy of 25 or 50 years, as defined under design service life, cannot be obtained with certain materials by reason of highly corrosive conditions, severe abrasive

Table 856.4

Thermoplastic Pipe Fill Height Tables

High Density Polyethylene (HDPE) Corrugated Pipe - Type S

Size (in)	Maximum Height of Cover (ft)
12	15
15	15
18	15
24	15
30	15
36	15
42	15
48	15
54	15
60	15

High Density Polyethylene (HDPE) Corrugated Pipe - Type C

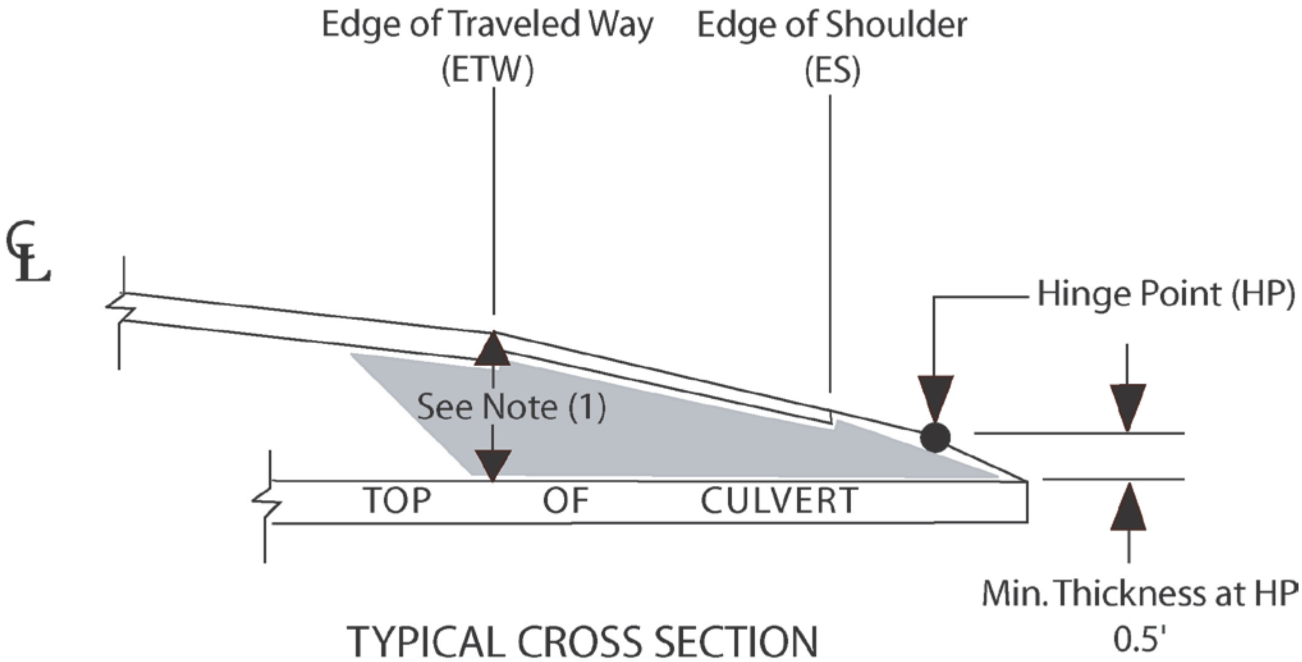
Size (in)	Maximum Height of Cover (ft)
12	5
15	5
18	5
24	5

Polyvinyl Chloride (PVC) Corrugated Pipe with Smooth Interior

Size (in)	Maximum Height of Cover (ft)
12	35
15	35
18	35
21	35
24	35
30	35
36	35

Table 856.5

Minimum Thickness of Cover for Culverts



MINIMUM THICKNESS OF COVER AT ETW							
Corrugated Metal Pipes and Pipe Arches	Steel Spiral Rib Pipe	Aluminum Spiral Rib Pipe, $S \leq 48"$	Aluminum Spiral Rib Pipe, $S > 48"$	Structural Plate Pipe	Reinforced Concrete Pipe (RCP) Under Rigid Pavement	RCP Under Flexible Pavement or Unpaved	Plastic Pipes
S/8 or 24" Min.	S/4 or 24" Min.	S/2 or 24" Min.	S/2.75 or 24" Min.	S/8 or 24" Min.	12" Min.	(Max Outside Dimension)/8 or 24" Min.	S/2 or 24" Min.

Notes:
(1) Minimum thickness of cover is measured at ultimate or failure edge of traveled way.
(2) Table is for HL-93 live load conditions only.
(3) ”S” in the table is the maximum inside diameter or span of a section.

conditions, or critical structural and construction requirements.

- For individual drainage systems such as roadway drainage systems or culverts which operate under hydrostatic pressure or culverts governed by hydraulic considerations and which would require separate design for each culvert type.
- When alterations or extensions of existing systems are required, the culvert type may be selected to match the type used in the existing system.

857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe

These instructions are general guidelines for alternative pipe culvert selection using the AltPipe computer program that is located on the Headquarters Division of Design alternative pipe culvert selection website at the following web address: http://www.dot.ca.gov/hq/oppd/altpipe.htm.

AltPipe is a web-based tool that may be used to assist materials engineers and designers in the appropriate selection of pipe materials for culvert and storm drain applications. The computations performed by AltPipe are based on the procedures and California Test Methods described in this

Chapter. AltPipe is not a substitute for the appropriate use of engineering judgment as conditions and experience would warrant. AltPipe establishes uniform procedures to assist the designer in carrying out the majority of the alternative pipe culvert selection functions of the Department, and is neither intended as, nor does it establish, a legal standard for these functions. Implementation of the results and output of this program is solely at the discretion of the user. The user is encouraged to first read the two informational links on the website titled 'Get More Information' and 'How to use Altpipe' prior to using the program.

Each alternative material selected for a drainage facility must provide the required design service life based on physical and structural factors, be of adequate size to satisfy the hydraulic design, and require the minimum of maintenance and construction cost for each site condition.

Step 1. Obtain the results of soil and water pH, resistivity, sulfate and chloride tests, proposed design life of culverts and make determination if any of the outfalls are in salty or brackish water. The Materials Report should include proposed design life and recommendations for pipe material alternatives. See Indexes 114.2 (3) and 114.3 (2).

Step 2. Obtain hydraulic studies and location data for pipe minimum sizes, and expected Q2-5 flow velocities. For pipes operating under outlet control, a critical element of pipe selection is the Manning's internal roughness value used in the hydraulic design. It is important to independently verify the roughness used in the design is applicable for the selected alternate materials from AltPipe. Rougher pipes may require larger sizes to provide adequate hydraulic capacities and need steeper slopes to produce desired cleaning velocities, usually however, pipe slope is maintained, and the only variable provided on the plans is pipe size.

Step 3. Determine the abrasion level from Table 852.2A from the maximum size of material that can be moved through a pipe, the expected Q2-5 flow velocities, and Table 855.2B. Field observations of channel bed material both upstream and downstream are recommended.

Step 4. Determine the maximum fill height.

Step 5. Using the AltPipe computer program that is located on the Headquarters Division of Design alternative pipe culvert selection website enter:

- Pipe diameter
- Maximum fill height
- Design service life
- pH
- Minimum resistivity
- Sulfate concentration
- Chloride concentration (for values greater than 2000, check boxes if end of culvert is exposed to brackish conditions and high tide line is below the crown of the culvert)
- Abrasion level
- 2-5-year Storm Flow Velocity (ft/sec)

Repeat step 5 as necessary and save each pipe in worksheet as needed and go to the final summary upon completion.

Table 857.2

Allowable Alternative Materials

Type of Installation	Service Life (yrs)[1]	Allowable Alternatives	Joint Type Standard	Joint Type Positive	Joint Type Downdrain
Culverts & Drainage Systems	50	ASSRP, ASRP, CAP, CASP, CSSRP, CIPCP, CSP, NRCP, SAPP, SSPP, SSRP, RCP, RCB, PPC	X	X	--
Overside Drains	50	CAP, CASP, CSP, PPC	--	--	X
Underdrains	50	PAP, PSP, PPET, PPVCP	X	--	--
Arches (Culverts & Drainage Systems)	50	ACSPA, CAPA, CSPA, RCA, SAPPA, SSPPA, SSPA	X	X	--

LEGEND

ACSPA	- Aluminized Corrugated Steel Pipe Arch
ASSRP	- Aluminized Steel Spiral Rib Pipe
ASRP	- Aluminum Spiral Rib Pipe
CAP	- Corrugated Aluminum Pipe
CAPA	- Corrugated Aluminum Pipe Arch
CSSRP	- Composite Steel Spiral Rib Pipe
CASP	- Corrugated Aluminized Steel Pipe, Type 2
CIPCP	- Cast-in-Place Concrete Pipe
CSP	- Corrugated Steel Pipe
CSPA	- Corrugated Steel Pipe Arch
NRCP	- Non-Reinforced Concrete Pipe
PAP	- Perforated Aluminum Pipe
PPC	- Plastic Pipe Culvert
PPET	- Perforated Polyethylene Tubing
PPVCP	- Perforated Polyvinyl Chloride Pipe
PSP	- Perforated Steel Pipe
RCA	- Reinforced Concrete Arch
RCB	- Reinforced Concrete Box
RCP	- Reinforced Concrete Pipe
SAPP	- Structural Aluminum Plate Pipe
SAPPA	- Structural Aluminum Plate Pipe Arch
SSPA	- Structural Steel Plate Arch
SSPP	- Structural Steel Plate Pipe
SSPPA	- Structural Steel Plate Pipe Arch
SSRP	- Steel Spiral Rib Pipe
X	- Permissible Joint Type for the Type of installation Indicated

NOTE:

1. The design service life indicated for the various types of installations listed in the table may be reduced to 25 years in certain situations. Refer to Index 855.1 for a discussion of service life requirements.

Step 6. The following alternatives are not included in AltPipe and will not be provided in the output Alternative pipe list: all non-circular shapes (arches, boxes, etc.), non reinforced concrete pipe (NRCP) and non-standard new products. Check Materials and Hydraulics reports and verify if any of these alternatives were recommended and supplement the AltPipe final summary accordingly. For reinforced concrete pipe (RCP), box (RCB) and arch (RCA) culverts, maintenance-free service life, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of exposed reinforcement at any point on the culvert. Changes in the design may be required in relatively severe acidic, chloride or sulfate environments. The levels of these constituents (either in the soil or water) will need to be identified in the project Materials or Geotechnical Design Report. The adopted procedure consists of a formula that the constituent concentrations are entered into in order to determine a pipe service life. The means for offsetting the affects of the corrosive elements is to increase the cover over the reinforcing steel, increase the cement content, or reduce the water/cement ratio.

Step 7. Table 855.2C constitutes a guide for abrasive resistant coatings in low to moderate abrasive conditions for metal pipe (i.e., Levels 1 through 5 in Table 855.2A) and is included in AltPipe. Table 855.2F constitutes a guide for minimum material thickness of abrasive resistant invert protection to achieve 50 years of maintenance-free service life in moderate to highly abrasive conditions (i.e., Levels 4 through 6 in Table 855.2A) and was not programmed into AltPipe. If pipe material thickness does not meet service life due to abrasive conditions, consideration for invert protection should be made using Table 855.2F as a guide.

857.3 Alternative Pipe Culvert (APC) and Pipe Arch Culvert List

Because of the difference in roughness coefficients between various materials, it may be necessary to specify a different size for each allowable material at any one location. In this event, it is recommended that the material with the smallest dimension be listed as the alternative size. Refer to Plans Preparation Manual for standard format to be used.

There may be situations where there is a different set of alternatives for the same nominal size of alternative drainage facilities. In this case the different sets of the same nominal size should be further identified by different types, for example, 18-inch alternative pipe culvert (Type A), 18-inch alternative pipe culvert (Type B), etc. No attempt to correlate type designation between projects is necessary. The first alternative combination for each culvert size on each project should be designated as Type A, second as Type B, etc.

Since the available nominal sizes for pipe arches vary slightly between pipe arch materials, it is recommended that the listed alternative pipe arch sizes conform to those sizes shown for corrugated steel pipe arches shown on Table 856.3D. The designer should verify the availability of reinforced concrete pipe arches. If reinforced concrete pipe arches are not available, oval shaped reinforced concrete pipe of a size necessary to meet the hydraulic requirements may be used as an alternative.

CHAPTER 860
ROADSIDE CHANNELS

Topic 861 – General

Index 861.1 - Introduction

Chapter 860 addresses the design of small open channels called roadside channels that are constructed as part of a highway drainage system. See Figure 861.1.

Figure 861.1
Small Roadside Channel

An open channel is a conveyance in which water flows with a free surface. Although closed conduits such as culverts and storm drains function as open channels when flowing partially full, the term is generally applied to natural and improved watercourses, gutters, ditches, and channels. While the hydraulic principles discussed in this chapter are valid for all drainage structures, the primary consideration is given to roadside channels.

In addition to performing its hydraulic function, the roadside channel should be economical to construct and maintain. Some roadside channels serve as dual purpose channels which concurrently function as infiltration swales for stormwater purposes. See Index 861.11, "Water Quality Channels". Roadside channel design should consider errant vehicles leaving the traveled way, be pleasing in appearance, convey collected water without damage to the transportation facility or adjacent property and minimize environmental impacts. These considerations are usually so interrelated that optimum conditions cannot be met for one without compromising one or more of the others. The objective is to achieve a reasonable balance, but the importance of traveler safety must not be underrated. See Index 861.4, "Safety Considerations".

Roadside channels play an important role in the highway drainage system as the initial conveyance for highway runoff. Roadside channels are often included as part of the typical roadway section. Therefore, the geometry of roadside channels depends on available right-of-way, flow capacity requirements, and the alignment and profile of the highway. Most roadside channels capture sheet flow from the highway pavement and cut slope and convey that runoff to larger channels or to culverts within the drainage system. See Figure 861.2.

Figure 861.2
Roadside Channel Outlet to Storm Drain at Drop Inlet

This initial concentration of runoff may create hydraulic conditions that are erosive to the soil that forms the channel boundary. To perform reliably, the roadside channel is often stabilized against erosion by placing a protective lining over the soil. This chapter presents two classes of channel linings called rigid and flexible linings that are well suited for construction of small roadside channels.

861.2 Hydraulic Considerations

An evaluation of hydraulic considerations for the channel design alternatives should be made early in the project development process. The extent of the hydrologic and hydraulic analysis should be commensurate with the type of highway, complexity of the drainage facility, and associated costs, risks, and impacts. Most of the roadside channels and swales discussed in this chapter convey design flows less than 50 cubic feet per second and generally do not require detailed hydrologic and hydraulic analyses beyond developing the parameters required for the Rational Formula (see Index 819.2(1)), Manning's Equation, and the shear stress equations presented within this Chapter and Hydraulic Engineering Circular (HEC) No. 15, "Design of Roadway Channels with Flexible Linings". The hydraulic design of an open channel consists of developing a channel section to carry the design discharge under the controlling conditions, adding freeboard as needed and determining the type of channel protection required to prevent erosion. In addition to erosion protection, channel linings can be used to increase the hydraulic capacity of the channel by reducing the channel roughness.

The hydraulic capacity of a roadside channel is dependent on the size, shape, slope and roughness of the channel section. For a given channel, the hydraulic capacity becomes greater as the grade or depth of flow increases. The channel capacity decreases as the channel surface becomes rougher. A rough channel can sometimes be an advantage on steep slopes where it is desirable to keep flow velocities from becoming excessively high. See Topics 866 and 867.

(1) *Flood Control Channels.* Flood control channels are typically administered by a local agency and present extreme consequences should failure occur. Therefore, when channels or drainage facilities under the jurisdiction of local flood control agencies or Corps of Engineers are involved, the design must be coordinated via negotiations with the District Hydraulic Engineer and the agencies involved. See Index 861.7, "Coordination with other Agencies" and Index 865.2.

For flood control purposes, a good open channel design within the right of way minimizes the effect on existing water surface profiles. Open channel designs which lower the water surface elevation can result in excessive flow velocities and cause erosion problems. A planned rise in water surface elevation can cause:

- Objectionable flooding of the roadbed and adjacent properties or facilities;
- An environmental and maintenance problem with sedimentation due to reduced flow velocities.

Additional hydraulic considerations may include: movable beds, heavy bedloads and bulking during flood discharges. A detailed discussion of sediment transport and channel morphology is contained in the FHWA's HDS No. 6 River Engineering for Highway Encroachments.

Reference is made to Volume VI of the AASHTO Highway Drainage Guidelines for a general discussion on channel hydraulic considerations.

861.3 Selection of "Design Flood"

As with other drainage facilities, the first step in the hydraulic design of roadside channels is to establish the range of peak flows which the channel section must carry. The recommended design flood and water spread criteria for roadway drainage type installations are presented in Table 831.3.

For flood control and cross drainage channels within the right of way, see Index 821.3, "Selection of Design Flood". Empirical and statistical methods for estimating design discharges are discussed in Chapter 810, "Hydrology".

861.4 Safety Considerations

An important aspect of transportation facility drainage design is that of traffic safety.

The shape of a roadside channel section should minimize vehicular impact and provide a traversable section for errant traffic leaving the traveled way. The ideal channel section, from a traversability standpoint, will have flattened side slopes and a curved transition to the channel bottom. When feasible, it is recommended that channels be constructed outside the clear recovery zone.

861.5 Maintenance Considerations

Design of open channels and roadside ditches should recognize that periodic maintenance inspection and repair is required. Provisions should be incorporated into the design for access to a channel by maintenance personnel and equipment. Consideration should be given to the size and type of maintenance equipment required when assessing the need for permanent or temporary access easements for entrance ramps and gates through the right of way fences.

Damaged channels can be expensive to repair and interfere with the safe and orderly movement of traffic.

Figure 861.3
Damaged Channel

Minor erosion damage within the right of way should be repaired immediately after it occurs and action taken to prevent the recurrence. Conditions which require extensive repair or frequently recurring maintenance may require a complete redesign rather than repetitive or extensive reconstruction. The advice of the District Hydraulics Engineer should be sought when evaluating the need for major restoration.

The growth of weeds, brush, and trees in a drainage channel can effectively reduce its hydraulic efficiency. See Figure 861.4. The result being that a portion of the design flow may overflow the channel banks causing flooding and possible erosion.

Figure 861.4
Concrete Lined Channel with Excessive Weed Growth

Accumulation of sediment and debris may destroy vegetative linings leading to additional erosion damage.

Channel work on some projects may be completed several months before total project completion. During this interim period, the contractor must provide interim protection measures. Per Index 865.3(3), the design engineer should include temporary channel linings to assure that minor erosion will not develop into major damage. As needed, the District Project Engineer may obtain vegetative recommendations from the District Landscape Architect. The Project Engineer must verify vegetative component compatibility with the final design.

861.6 Economics

Economical drainage design is achieved by selecting the design alternative which best satisfies the established design criteria at the lowest cost.

The economic evaluation of design alternatives should be commensurate with the complexity and importance of the facility. Analysis of the channel location, shape, size, and materials involved may reveal possibilities for reducing construction costs, flood damage potential, maintenance problems and environmental impacts.

861.7 Coordination with Other Agencies

There are many Federal, State and local agencies and private entities engaged in water related planning, construction and regulation activities whose interests can affect the design of some highway drainage channels (e.g., flood control channels described under Index 861.2(1)). Such agencies may request the channel design satisfy additional and perhaps governing design criteria. Early coordination with these agencies may help avoid delays in the project development process and post-project conflicts. Early coordination may also reveal opportunities for cooperative projects which may benefit both Caltrans and the water resources agency. For information on cooperative agreements refer to Index 803.2.

861.8 Environment

Many of the same principles involved in sound highway construction and maintenance of open channels parallel environmental considerations. Environmental problems can arise if riparian species inhabit the channel. Erosion, sedimentation, water quality, and aesthetics should be of prime concern to the highway design engineer. Refer to Index 110.2 and the Project Planning and Design Guide for discussion on control of water pollution.

861.9 Unlined Channels

Whenever feasible, roadside channels should be designed with natural bottoms. Use linings only when warranted.

Refer to Table 865.2 for typical permitted shear stress and velocity for bare soil and vegetation.

861.10 Lined Channels

The main purposes of channel linings are:

(a) To prevent erosion damage.

(b) To increase velocity for prevention of excessive sedimentation

(c) To increase capacity.

See Topic 865 for design concepts.

861.11 Water Quality Channels

Biofiltration swales are vegetated channels, typically configured as trapezoidal or v-shaped channels (trapezoidal recommended where feasible) that receive and convey stormwater flows while meeting water quality criteria and other flow criteria independent of Chapter 860. Pollutants are removed by filtration through the vegetation, sedimentation, absorption to soil particles, and infiltration through the soil. Strips and swales are effective at trapping litter, total suspended solids (soil particles), and particulate metals. In most cases, flow attenuation is also provided.

Refer to Appendix B, Table B-1 of the Project Planning and Design Guide for a summary of preliminary design factors for biofiltration strips and swales:

http://www.dot.ca.gov/hq/oppd/stormwtr/ppdg/swdr2012/PPDG-May-2012.pdf

See HDM Table 816.6A and Index 865.5 for Manning’s roughness coefficients used for travel time calculations for the rational formula based on water quality flow (WQF) to check swale performance against biofiltration criteria at WQF, i.e., a Hydraulic Residence Time of 5 minutes or more; a maximum velocity of 1.0 ft/s; and a maximum depth of flow of 0.5 ft. See Bio-Strips and Bio-Swales under Biofiltration Design Guidance at:

http://www.dot.ca.gov/hq/oppd/storm1/caltrans_20090729.html

861.12 References

More complete information on hydraulic principles and engineering techniques of open channel design may be found in FHWA's Hydraulic Design Series No. 3, "Design Charts for Open Channel Flow", Hydraulic Design Series No. 4, "Introduction to Highway Hydraulics", Hydraulic Engineering Circular No. 15 (HEC No. 15), “Design of Roadway Channels with Flexible Linings” and Hydraulic Engineering Circular No. 22 (HEC No. 22), Chapter 5, “Urban Drainage Design Manual – Roadside and Median Channels”. For a general textbook discussion of open channel hydraulics, reference is made to "Open-Channel Hydraulics" by Ven Te Chow. In addition, many helpful design aids are included in "Handbook of Hydraulics", by Brater and King.

Topic 862 - Roadside Drainage Channel Location

862.1 General

Assuming adequate functional design, the next most important design consideration is channel location. Locations that avoid poorly drained areas, unstable soil conditions, and frequently flooded areas can greatly reduce drainage related problems. Refer to Index 110.4 for discussion on wetlands protection.

Typically drainage and open channel considerations are not considered the primary decision factors in the roadway location; however they are factors which will often directly or indirectly affect many other considerations. Often minor alignment adjustments can avoid serious drainage problems.

If a channel can be located far enough away from the highway, the concerns of traffic safety and aesthetics can be significantly mitigated. See Figure 862.1. The cost of additional right of way may be offset somewhat by the reduced cost of erosion control, traffic protection, and landscaping.

Figure 862.1
Small-Rock Lined Channel
Outside of Clear Recovery Zone

862.2 Alignment and Grade

Ordinarily, the highway drainage channel must be located where it will best serve its intended purpose, using the grade and alignment obtainable at the site. Insofar as practicable, abrupt changes in alignment and grade should be avoided. A sharp change in alignment presents a point of attack for flowing water, and abrupt changes in grade can result in possible scour when the grade is steepened or deposition of transported material when the grade is flattened.

Ideally, a drainage channel should have flow velocities that neither erode nor cause deposition in the channel. This optimum velocity is dependent on the size and slope of channel, the quantity of flowing water, the material used to line the channel, the nature of the bedding soil and the sediment being transported by the flow. Refer to Table 865.2 for recommended permissible flow velocities in unlined channels.

Realignment considerations for channels within the right of way are discussed in Index 867, Channel Changes.

862.3 Point of Discharge

The point of discharge into a natural watercourse requires special attention. Water entering a natural watercourse from a highway drainage channel should not cause eddies with attendant scour of the natural watercourse. In erodible embankment soils, if the flow line of the drainage channel is appreciably higher than that of the watercourse at the point of discharge, then the use of a spillway may be advisable to prevent erosion of the channel.

Topic 863 - Channel Section

863.1 Roadside and Median Channels

Roadside and median channels are open-channel systems which collect and convey stormwater from the pavement surface, roadside, and median areas. These channels may outlet to a storm drain piping system via a drop inlet (see Figure 861.2), to a detention or retention basin or other storage component, or to an outfall channel. Roadside and median channels are normally triangular or trapezoidal in cross section and are lined with grass or other protective lining.

Reference is made to the FHWA publication HEC No. 22, Chapter 5.

The shape of a channel section is generally determined by considering the intended purpose, terrain, flow velocity and quantity of flow to be conveyed.

863.2 Triangular

The triangular channel or V-ditch is intended primarily for low flow conditions such as in median and roadside ditches. V-shaped ditches are susceptible to erosion and will require lining when shear stress and velocity exceed the values given for bare soil in Table 865.2. It is good practice to round the bottom of a V-ditch. See Figure 862.1 and Figure 863.1.

Figure 863.1
Small-Rock Lined Triangular Channel with Rounded Bottom

863.3 Trapezoidal

The most common channel shapes is the trapezoidal section.

Trapezoidal channels are easily constructed by machinery and are often the most economical.

When a wide trapezoidal section is proposed, both traffic safety and aesthetics can be improved by rounding all angles of the channel cross section with vertical curves. The approximate length of these vertical curves can be determined by the formula:

$$L = \frac{40}{X}$$

where:

L = Length of vertical curve in feet

X = Horizontal component of side slopes expressed as x, y coordinates with y = 1

For narrow channels, L, is limited to the bottom width.

863.4 Rectangular

Rectangular channels are used to convey large flows in areas with limited right of way. At some locations, guardrail or other types of positive traffic barrier may be necessary between the traveled way and the channel.

Though rectangular channels are relatively expensive to construct, since the walls must be designed as earth retaining structures, the construction costs can be somewhat offset by the reduced costs associated with right of way, materials, and channel excavation. See Index 865.2 for the design of concrete lined flood control channels.

Topic 864 - Channel Stability Design Concepts

864.1 General

The gradient of roadside channels typically parallels the grade of the highway. Even at relatively mild highway grades, highly erosive hydraulic conditions can exist in adjacent roadside channels. Consequently, designing a stable conveyance becomes a critical component in the design of roadside channels.

The need for erosion prevention is not limited to the highway drainage channels; it extends throughout the right-of-way and is an essential feature of adequate drainage design. Erosion and maintenance are minimized largely by the use of flat side slopes rounded and blended with natural terrain, drainage channels designed with due regard to location, width, depth, slopes, alignment, and protective treatment, proper facilities for groundwater interception, dikes, berms, and other protective devices, and protective ground covers and planting.

864.2 Stable Channel Design Procedure

For most highway drainage channels bed and side slope instability cannot be tolerated and stable channel design must be based on the concepts of static equilibrium, including the use of a lining material if necessary. The permissible tractive force (shear stress) procedure requires that the shear stresses on the channel bottom and sides do not exceed the allowable amounts for the given channel boundary. Based on the actual physical processes involved in maintaining a stable channel, specifically the stresses developed at the interface between flowing water and materials forming the channel boundary, the tractive force procedure is a more realistic model and was adopted as the preferred design procedure for HEC No. 15, which is the primary reference for stable channel design.

The maximum shear stress along the channel bottom may be estimated by the following equation:

$$\tau_d = \gamma d S$$

where:

τ_d = Shear stress in channel at maximum depth, lb/ft^2

γ = Specific weight of water

d = Maximum depth of flow in channel for the design discharge, ft

S = Slope of channel, ft/ft

When the permissible shear stress is greater than or equal to the computed shear stress, the lining is considered acceptable:

$$\tau_p \geq SF\tau_d$$

where:

τ_p = Permissible shear stress for the channel lining, lb/ft

SF = Safety factor

The safety factor provides for a measure of uncertainty, as well as a means for the designer to reflect a lower tolerance for failure by choosing a higher safety factor. A safety factor of 1.0 is appropriate in many cases and may be considered the default. However, safety factors from 1.0 to 1.5 may be appropriate, subject to the designer's discretion, where one or more of the following conditions may exist:

(a) critical or supercritical flows are expected

(b) climatic regions where vegetation may be uneven or slow to establish

(c) significant uncertainty regarding the design discharge

(d) consequences of failure are high

The relationship between permissible shear stress and permissible velocity for a lining can be found by substituting the equation for maximum shear stress and continuity equation into Manning's equation:

$$V_p = \frac{\alpha}{n\sqrt{\gamma d}} R^{1/6} \tau_p^{1/2}$$

where:

V_p = Permissible velocity, ft/s

τ_p = Permissible shear stress, lb/ft^2

α = Unit conversion constant, 1.49

As a guide, Table 865.2 provides typical values of permissible velocity and permissible shear stress for selected lining types.

The basic procedure for designing a flexible lining consists of the following steps.

Step 1. Determine a design discharge, Q, and select the channel slope and channel shape.

Step 2. Select a trial lining type. Initially, the Engineer may need to determine if a long-term lining is needed and whether or not a temporary or transitional lining is required. For determining the latter, the trial lining type could be chosen as the native material (unlined), typically bare soil. For example, it may be determined that the bare soil is insufficient for a long-term solution, but vegetation is a good solution. For the transitional period between construction and vegetative establishment, analysis of the bare soil will determine if a temporary lining is prudent. Per Index 865.1, District Landscape should be consulted to provide feasible long-term vegetation recommendations. The Engineer and the Landscape Architect should discuss the compatibility of any living materials (temporary, transitional or permanent) with the proposed lining material and verify impacts to conveyance before the Engineer finalizes the design.

Step 3. Estimate the depth of flow, d_i in the channel and compute the hydraulic radius, R. The estimated depth may be based on physical limits of the channel, but this first estimate is essentially a guess. Iterations on Steps 3 through 5 may be required.

Step 4. Estimate Manning's n and the discharge implied by the estimated n and flow depth values. Calculate the discharge (Q_i).

Step 5. Compare Q_i with Q. If Q_i is within 5 percent of the design, Q, then proceed on to Step 6. If not, return to Step 3 and select a new estimated flow depth, d_{i+1}. This can be estimated from the following equation or any other appropriate method.

$$d_{i+1} = d_i \left(\frac{Q}{Q_i}\right)^{0.4}$$

Step 6. Calculate the shear stress at maximum depth, τ_d, determine the permissible shear stress, τ_p, according to the methods described in HEC No. 15 and select an appropriate safety factor (i.e., 1 to 1.5).

Step 7. Compare the permissible shear stress to the calculated shear stress from Step 6 using:

$$\tau_p \geq SF\tau_d$$

If the permissible shear stress is adequate then the lining is acceptable. If the permissible shear is inadequate, then return to Step 2 and select an alternative lining type with greater permissible shear stress from Table 865.2. As an alternative, a different channel shape may be selected that results in a lower depth of flow. The selected lining is stable and the design process is complete. Other linings may be tested, if desired, before specifying the preferred lining.

Direct solutions for Manning's equation for many channels of trapezoidal, rectangular, triangular and circular cross sections can be found within the Channel Analysis subcomponent FHWA's Hydraulic Toolbox software program.

864.3 Side Slope Stability

Shear stress is generally reduced on the channel sides compared with the channel bottom. The maximum shear on the side of a channel is given by the following equation:

$$\tau_s = K_1\tau_d$$

where:

τ_s = Side shear stress on the channel, lb/ft^2

K_1 = Ratio of channel side to bottom shear stress

τ_d = Shear stress in channel at maximum depth, lb/ft^2

The value K_1 depends on the size and shape of the channel. For parabolic or V-shape with rounded bottom channels there is no sharp discontinuity along the wetted perimeter and therefore it can be assumed that shear stress at any point on the side slope is related to the depth at that point using the shear stress equation from Index 864.2:

$$\tau_d = \gamma dS$$

For trapezoidal and triangular channels, the following K_1 values may be applied:

K_1 = 0.77 $\quad Z \leq 1.5$

K_1 = 0.066Z + 0.67 $\quad 1.5 < Z < 5$

K_1 = 1.0 $\quad 5 \leq Z$

The Z value represents the horizontal dimension 1:Z (V:H). Use of side slopes steeper than 1:3 (V:H) is not encouraged for flexible linings because of the potential for erosion of the side slopes. Steep side slopes are allowable within a channel if cohesive soil conditions exist. Channels with steep slopes should not be allowed if the channel is constructed in non-cohesive soils.

For channels lined with gravel or small-rock slope protection, the maximum suggested side slope is 1 V : 3 H, and flatter slopes are encouraged. If steeper side slopes are required, see Chapter 6 of HEC No. 15 for design procedures.

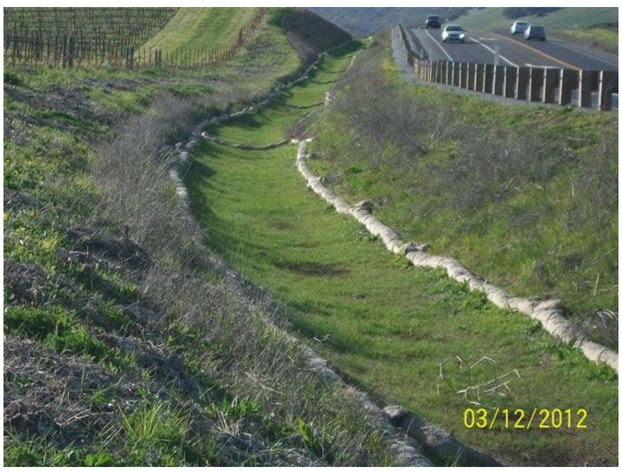
Topic 865 - Channel Linings

865.1 Flexible Verses Rigid

Lining materials may be classified as flexible or rigid. Flexible linings are able to conform to changes in channel shape and can sustain such changes while maintaining the overall integrity of the channel. In contrast, rigid linings cannot change shape and tend to fail when a portion of the channel lining is damaged. Channel shape may change due to frost-heave, slumping, piping, etc. Typical flexible lining materials include grass or small-rock slope protection, while typical rigid lining materials include hot mixed asphalt or Portland cement concrete. Flexible linings are generally less expensive, may have a more natural appearance, permit infiltration and exfiltration and are typically more environmentally acceptable. Vegetative channel lining is also recognized as a best management practice for storm water quality design in highway drainage systems. A vegetated channel helps to deposit highway runoff contaminants (particularly suspended sediments) before they leave the highway right of way and enter streams. See Index 861.11 ‘Water Quality Channels’ and Figure 865.1.

On steep slopes, most vegetated flexible linings are limited in the erosive forces they can sustain without damage to the channel and lining unless the vegetative lining is combined with another more erosion-resistant long-term lining below, such as a cellular soil confinement system. See Figure 865.1 and Index 865.3(1). The District Landscape Architect should be contacted to provide viable vegetation alternatives within the District, however all design responsibilities belong to the Project Engineer.

Figure 865.1
Steep-Sloped Channel with Composite Vegetative Lining



Vegetative flexible lining placed on top of cellular soil confinement system on a steep-sloped channel.

865.2 Rigid

A rigid lining can typically provide higher capacity and greater erosion resistance and in some cases may be the only feasible alternative.

Rigid linings are useful in flow zones where high shear stress or non-uniform flow conditions exist, such as at transitions in channel shape or at an energy dissipation structure.

The most commonly used types of rigid lining are hot mixed asphalt and Portland cement concrete. Hot mixed asphalt is used mainly for small ditches, gutters and overside drains (see Standard Plan D87D) because it cannot withstand hydrostatic pressure from the outside.

Table 865.1 provides a guide for Portland cement concrete and air blown mortar roadside channel linings. See photo below Table 865.1 for example.

For the design of concrete lined flood control channels discussed in Index 861.2 (1), see U.S. Army Corps of Engineers publication; "Structural Design of Concrete Lined Flood Control Channels", EM 1110-2-2007:

http://planning.usace.army.mil/toolbox/library/EMs/em1110.2.2007.pdf

Table 865.1
Concrete(2) Channel Linings

Abrasion Level(1)	Thickness of Lining (in)		Minimum Reinforcement
	Sides	Bottom	
1 - 3	5	5	6 x 6-W2.9 x W2.9 welded wire fabric

NOTES:

(1) See Table 855.2A.

(2) Portland Cement Concrete or Air Blown Mortar

Figure 865.2
Concrete Lined Channel

For large flows, consideration should be given to using a minimum bottom width of 12 feet for construction and maintenance purposes, but depths of flow less than one foot are not recommended.

Despite the non-erodible nature of rigid linings, they are susceptible to failure from foundation instability and abrasion. The major cause of failure is undermining that can occur in a number of ways.

865.3 Flexible

Flexible linings can be long-term, transitional or temporary. Long-term flexible linings are used where the channel requires protection against erosion for the design service life of the channel. Per Index 861.12, more complete information on hydraulic principles and engineering techniques of flexible channel lining design may be found in HEC No. 15 and Chapter 5 of HEC No. 22.

Flexible linings act to reduce the shear stress on the underlying soil surface. Therefore, the erodibility of the underlying soil is a key factor in the performance of flexible linings. Erodibility of non-cohesive soils (plasticity index less than 10) is mainly due to particle size, while cohesive soil erodibility is a function of cohesive strength and soil density. Vegetative and rolled erosion control product lining performance relates to how well they protect the underlying soil from shear stress, and so these lining types do not have permissible shear stresses independent of soil type. The soil plasticity index should be included in the Materials or Geotechnical Design Report.

In general, when a lining is needed, the lowest cost lining that affords satisfactory protection should be used. This may include vegetation used alone or in combination with other types of linings. Thus, a channel might be grass-lined on the flatter slopes and lined with more resistant material on the steeper slopes. In cross section, the channel might be lined with a highly resistant material (e.g., cellular soil confinement system – see Index 865.3(1) *Long Term*) within the depth required to carry floods occurring frequently and lined with grass above that depth for protection from the rare floods.

(1) Long Term. Long-term lining materials include vegetation, rock slope protection, gabions (wire-enclosed rock), and turf reinforcement mats with enhanced UV stability. Standard Specification Section 72-4 includes specifications for constructing small-rock slope protection for gutters, ditches or channels and includes excavating and backfilling the footing trench, placing RSP fabric and placing small rocks (cobble, gravel, crushed gravel, crushed rock, or any combination of these) on the slope. Where the channel design includes a requirement for runoff infiltration to address stormwater needs, the designer may need to consider installation of a granular filter in lieu of RSP fabric if it is anticipated that the RSP fabric would become clogged with sediment. See following link to HEC No. 23, Volume 2, Design Guideline 16, Index 16.2.1, for information on designing a granular filter:

http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09112/page16.cfm

Standard Specification Section 72-16 includes specifications for constructing gabion structures. Gabions consist of wire mesh baskets that are placed and then filled with rock. Gabion basket wires are susceptible to corrosion and are most appropriate for use as a channel lining where corrosion potential is minimized, such as desert or other arid locations.

Cellular soil confinement systems may be used as an alternative for steep channels with a variety of infills available including soil and gravel. Soil confinement systems consist of sheet polyethylene spot welded to form a system of individual confinement cells. See Figure 865.3.

Figure 865.3
Long-Term Flexible Lining

Placing a polyethylene cellular soil confinement system on a steep-sloped channel.

Per Index 865.1, these systems may be combined with other vegetated flexible linings, e.g., turf reinforcement mats.

(2) *Transitional.* Transitional flexible linings are used to provide erosion protection until a long-term lining, such as grass, can be established. For mild slopes, these may include jute netting (depending on environmental, i.e., wildlife, parameters) or turf reinforcement. Turf reinforcement can serve either a transitional or long-term function by providing additional structure to the soil/vegetation matrix. Typical turf reinforcement materials include gravel/soil mixes and turf reinforcement mats (TRM's). A TRM is a non-degradable rolled erosion control product (RECP) processed into a three-dimensional matrix. For examples see following link:

http://www.dot.ca.gov/hq/LandArch/ec/recp/trm.htm

The design for transitional products should be based on a flood event with an exceedance probability at least equal to the expected product service life (i.e., 12 to 36 months).

(3) *Temporary.* Temporary channel linings are used without vegetation to line channels that might be part of a construction site or some other short-term channel situation.

Standard Specification Section 21-1 was developed primarily to address slope erosion products, however, it includes specifications for constructing turf reinforcing mats, netting and rolled erosion control products (RECP's – see Index 865.6) which may also be applied to channels as temporary and transitional linings. See Index 865.1for coordinating vegetative recommendation with District Landscape Architecture.

865.4 Composite Lining Design

The procedure for composite lining design is based on the stable channel design procedure presented in Index 864.2 with additional sub-steps to account for the two lining types. Specifically, the modifications are:

Step 1. Determine design discharge and select channel slope and shape. (No change.)

Step 2. Need to select both a low flow and side slope lining. (See Table 866.3A.)

Step 3. Estimate the depth of flow in the channel and compute the hydraulic radius. (No change.)

Step 4. After determining the Manning's n for the low flow and side slope linings, calculate the effective Manning's n:

$$n_e = \left[\frac{P_L}{P} + \left(1 - \frac{P_L}{P}\right)\left(\frac{n_s}{n_L}\right)^{3/2}\right]^{2/3} n_L$$

where:

n_e = Effective Manning's n value for the composite channel

P_L = Low flow lining perimeter, ft

P = Total flow perimeter, ft

n_s = Manning's n value for the side slope lining

n_L = Manning's n value for the low flow lining

Step 5. Compare implied discharge and design discharge. (No change.)

Step 6. Determine the shear stress at maximum depth, τ_d ($\tau_d = \gamma dS$), and the shear stress on the channel side slope, τ_s (see Index 864.2).

Step 7. Compare the shear stresses, τ_d and τ_s, to the permissible shear stress, τ_p, for each of the channel linings. If τ_d or τ_s is greater than the τ_p for the respective lining, a different combination of linings should be evaluated. See Table 865.2.

865.5 Bare Soil Design and Grass Lining

Per Index 865.1, the District Landscape Architect should be contacted to recommend vegetation alternatives (including vegetation for transitional products, if needed) and the same procedure for the stable channel design procedure presented in Index 864.2 should be followed by the Project Engineer. See Figure 865.4 for grass lining example in a median channel. For slope stability when constructing embankment (4:1 and steeper), 85-90% relative compaction is desired. Although not optimal for best plant growth, compaction of up to 90% is not a major constraint for grass establishment. Prior to seeding, scarification to a depth of 1 inch of the compacted soil surface is recommended for improving initial runoff absorption and ensuring the seed is incorporated into the soil. A temporary degradable erosion control blanket (ECB) (e.g., single net straw) can then be installed on top.

The permissible shear stress for the vegetation is based on the design flood (Table 831.3). If the calculated shear for any given vegetation method is inadequate, then an alternative vegetation type with greater shear stress must be selected and/or a different channel shape may be selected that results in a lower depth of flow.

Figure 865.4
Grass-Lined Median Channel

The permissible shear stress for rolled erosion control products should be based on a flood event with an exceedance probability no less than the expected product service life (i.e., 12 to 36 months). The maximum shear stresses for channel applications shown in Erosion Control Technology Council Rolled Erosion Control Products Specification Chart must be lower than the permissible shear stresses indicated in Table 865.2. See: http://www.ectc.org/specifications.asp

The Manning's roughness coefficient for grass linings varies depending on grass properties and shear stress given that the roughness changes as the grass stems bend under flow. The equation describing the n value for grass linings is:

$$n = \alpha C_n \tau_0^{-0.4}$$

where:

τ_0 = Average boundary shear stress, lb/ft^2

α = Unit conversion constant, 0.213

C_n = Grass roughness coefficient (use 0.20 or Tables 4.3 and 4.4 from HEC-15)

The remaining shear at the soil surface is termed the effective shear stress. When the effective shear stress is less than the allowable shear for the soil surface, then erosion of the soil surface will be controlled. The effective shear at the soil surface is given by the following equation.

$$\tau_e = \tau_d \left(1 - C_f\right) \left(\frac{n_s}{n}\right)^2$$

where:

τ_e = Effective shear stress on the soil surface, lb/ft^2

τ_d = Design shear stress, lb/ft^2

C_f = Grass cover factor (use 0.6 to 0.8 or Table 4.5 from HEC-15)

n_s = Soil grain roughness

n = Overall lining roughness

The soil grain roughness, n_s, is 0.016 when D_{75} < 0.05 in. For larger grained soils the soil grain roughness is

$$n_s = \alpha (D_{75})^{1/6}$$

where:

n_s = Soil grain roughness (D_{75} > 1.3 (0.05 in))

D_{75} = Soil size where 75 percent of the material is finer, in

α = Unit conversion constant, 0.026

The permissible soil shear stress for fine-grained, non-cohesive soils (D_{75} < 0.05 in. is relatively constant and is conservatively estimated at 0.02 lb/ft^2. For coarse grained, non-cohesive soils (0.05 in. < D_{75} < 2 in.) the following equation applies.

$$\tau_{p,soil} = \alpha D_{75}$$

where:

$\tau_{p,soil}$ = Permissible soil shear stress, lb/ft^2

D_{75} = Soil size where 75 percent of the material is finer, in

α = Unit conversion constant, 0.4

Table 865.2[2]
Permissible Shear and Velocity for Selected Lining Materials

Boundary Category	Boundary Type	Permissible Shear Stress (lb/ft^2)	Permissible Velocity (ft/s)
Soils[1]	Fine colloidal sand	0.03	1.5
	Sandy loam (noncolloidal)	0.04	1.75
	Clayey sands (cohesive, PI ≥ 10)	0.095	2.6
	Inorganic silts (cohesive, PI ≥ 10)	0.11	2.7
	Silty Sands (cohesive, PI ≥ 10)	0.072	2.4
	Alluvial silt (noncolloidal)	0.05	2
	Silty loam (noncolloidal)	0.05	2.25
	Finer than course sand - D_{75} < 0.05 in. (non-cohesive)	0.02	1.3
	Firm loam	0.075	2.5
	Fine gravels	0.075	2.5
	Fine gravel (non-cohesive, D_{75} = 0.3 in, PI<10)	0.12	2.8
	Gravel (D_{75} = 0.6 in) (non-cohesive, D_{75} = 0.6 in, PI<10)	0.24	3.7
	Inorganic clays (cohesive, PI ≥ 20)	0.14	2.9
	Stiff clay	0.25	4.5
	Alluvial silt (colloidal)	0.25	3.75
	Graded loam to cobbles	0.38	3.75
	Graded silts to cobbles	0.43	4
	Shales and hardpan	0.67	6
Vegetation	Class A turf (Table 4.1, HEC No. 15)	3.7	8
	Class B turf (Table 4.1, HEC No. 15)	2.1	7
	Class C turf (Table 4.1, HEC No. 15)	1.0	3.5
	Long native grasses	1.7	6
	Short native and bunch grass	0.95	4

Table 865.2(2) (con't.)
Permissible Shear and Velocity for Selected Lining Materials

Boundary Category	Boundary Type	Permissible Shear Stress (lb/ft^2)	Permissible Velocity (ft/s)
Rolled Erosion Control Products (RECPs)			
Temporary Degradable Erosion Control Blankets (ECBs)	Single net straw	1.65	3
	Double net coconut/straw blend	1.75	6
	Double net shredded wood	1.75	6
Open Weave Textile (OWT)	Jute	0.45	2.5
	Coconut fiber	2.25	4
	Vegetated coconut fiber	8	9.5
	Straw with net	1.65	3
Non Degradable Turf Reinforcement Mats (TRMs)	Unvegetated	3	7
	Partially established	6.0	12
	Fully vegetated	8.00	12
Rock Slope Protection, Cellular Confinement and Concrete			
Rock Slope Protection	Small-Rock Slope Protection (4-inch Thick Layer)	0.8	6
	Small-Rock Slope Protection (7-inch Thick Layer)	2	8
	No. 2	2.5	10
	Facing	5	12
Gabions	Gabions	6.3	12
Cellular Confinement: Vegetated infill	71 in^2 cell and TRM	11.6	12
Cellular Confinement: Aggregate Infill	1.14 - in. D_{50} (45 in^2 cell)	6.9	12
	3.5" D_{50} (45 in^2 cell)	15.1	11.5
	1.14" D_{50} (71 in^2 cell)	13.2	12
	3.5" D_{50} (71 in^2 cell)	18	11.7
	1.14" D_{50} (187 in^2 cell)	10.92	12
	3.5" D_{50} (187 in^2 cell)	10.55	12
Cellular Confinement: Concrete Infill	(71 in^2 cell)	2	12
Hard Surfacing	Concrete	12.5	12

NOTES:

(1) PI = Plasticity Index (From Materials or Geotechnical Design Report)

(2) Some materials listed in Table 856.2 have been laboratory tested at shear stresses/velocities above those shown. For situations that exceed the values listed for roadside channels, contact the District Hydraulic Engineer.

A simplified approach for estimating the permissible shear stress for cohesive soils (based on Equation 4.6 in Chapter 4 of HEC No. 15) is illustrated in Figure 4.1 of Chapter 4 in HEC No. 15. The combined effects of the soil permissible shear stress and the effective shear stress transferred through the vegetative lining results in a permissible shear stress for the given conditions. Table 865.2 provides typical values of permissible shear stress and permissible velocity for cohesive soils and selected lining types. Representative values for different soil, vegetation and lining types are based on the methods found in Chapter 4 of HEC No. 15 while those for gravel, rock gabions and rock slope protection are based on methods found in Chapters 6 and 7 of HEC No. 15. The permissive shear stress values shown for soil confinement systems are based on testing by others, however, the maximum permissive velocity shown in Table 865.2 for all boundary types has been limited to 12 feet per second based on the following assumptions:

- The upper limit of flow rate is 50 cfs
- The longitudinal slope is 10 percent maximum
- The maximum side slope is 2H:1V
- The maximum storm duration is one hour

When the permissible shear stress is greater than or equal to the computed shear stress, the lining is considered acceptable. If the computed velocity exceeds the permissive velocity, or any of the above-listed assumptions are exceeded, contact the District Hydraulic Engineer for support.

865.6 Rolled Erosion Control Products

(1) General. Manufacturers have developed a variety of rolled erosion control products (RECPs) for erosion protection of channels.

RECPs consist of materials that are stitched or bound into a fabric. Vegetative and RECP lining performance relates to how well they protect the underlying soil from shear stresses so these linings do not have permissible shear stresses independent of soil types. Chapters 4 (vegetation) and 5 (RECPs) of HEC No. 15 describe the methods for analyzing these linings. Standard Specification Section 21-1 was developed primarily to address slope erosion products, however, the specifications for constructing turf reinforcing mats (TRM's), open weave textiles and erosion control blankets may also be applied to channels as temporary and transitional linings, and some TRM's may be used as permanent linings.

(2) Non-Hydraulic Design Considerations. The long-term performance of TRMs has traditionally been evaluated using hydraulic testing performance within controlled flume environments, or laboratory testing of specific parameters, usually conforming to ASTM or other industry standards. In recent years additional important design factors have been identified, from damages due to insect infestation to drainage problems or soil conditions resulting in poor vegetative establishment. Table 5.5 within Chapter 5 of HEC No. 15 provides a detailed TRM protocol checklist.

Six broad categories of stressors or potential damages to RECPs are listed below that can cause decrease in performance, considered as a function of specific properties of these lining materials.

(a) Environmental stress – tensile stresses that exceed the mechanical strength of the material accelerated by other stresses in the exposure environment.

Many manufacturer-reported values for maximum velocity or shear stress are based on short duration testing, however, longer duration flows – hours to days – more closely represent field conditions. Erosive properties of soils change with saturation, vegetation becomes stressed or damaged, and properties of some lining materials change with long periods of inundation or hydraulic stress. The result is that maximum reported shear stress and velocity may overestimate actual field performance of the full range of channel lining materials in the event of longer duration flows (Table 865.2). See Index 865.5 for safety factor discussion.

(b) Mechanical damage – localized damage due to externally applied loads such as debris or machinery, often during installation but also due to operation and maintenance activities

(c) Oxidation – due to exposure to air and water, a chemical reaction with a specific chemical group in a constituent polymer that leads to damage at a molecular level and changes in physical properties. Other chemical stresses can include acidity, corrosives, salinity, ozone and other air pollutants.

(d) Photo degradation – change in chemical structure due to exposure to UV wavelengths of sunlight, most often occurring during installation, prior to full vegetation establishment or inadequate vegetation establishment and coverage over time.

UV-Resistance per ASTM D-4355 should conform to the following for the specified type of TRM and design life:

- Temporary or transitional TRM – 90% tensile strength retained at 500 hr for the TRM product to be considered up to a 5-year design life.
- Long-term TRM – 90% tensile strength retained at 5,000 hr for the TRM product to be considered up to a 50-year design life.

(e) Temperature instability – changes in appearance, weight, dimension or other properties as a result of low, high, or cyclic temperature exposure.

As TRM or other materials are degrading, the vegetative component of a project is simultaneously becoming established, presumably leading to an overlap in effectiveness of each component. The engineer must carefully evaluate published performance data for specific materials with anticipated degradation, consider specific performance added by vegetative components, and apply a factor of safety in choosing materials that may provide enough strength initially to bridge the gap. Per Index 865.6(1), the District Landscape Architect should be consulted to provide viable long-term and compatible transitional vegetation recommendations (if required by the designer).

Topic 866 - Hydraulic Design of Roadside Channels

866.1 General

Open channel hydraulic design is of particular importance to highway design because of the interrelationship of channels to most highway drainage facilities.

The hydraulic principles of open channel flow are based on steady state uniform flow conditions, as defined in Index 866.2. Though these conditions are rarely achieved in the field, generally the variation in channel properties is sufficiently small that the use of uniform flow theory will yield sufficiently accurate results for most roadside channels.

866.2 Flow Classifications

(1) Steady vs. Unsteady Flow. The flow in an open channel can be classified as steady or unsteady. The flow is said to be steady if the depth of flow at a section, for a given discharge, is constant with respect to time. The flow is considered unsteady if the depth of flow varies with respect to time.

(2) Uniform Flow. Steady flow can further be classified as uniform or nonuniform. The flow is said to be uniform if the depth of flow and quantity of water are constant at every section of the channel under consideration. Uniform flow can be maintained only when the shape, size, roughness and slope of the channel are constant. Under uniform flow conditions, the depth and mean velocity of flow is said to be normal. Under these conditions the water surface and flowlines will be parallel to the stream bed and a hydrostatic pressure condition will exist, the pressure at a given section will vary linearly with depth.

As previously mentioned, uniform flow conditions are rarely attained in the field, but the error in assuming uniform flow in a channel of fairly constant slope, roughness and cross section is relatively small when compared to the uncertainties of estimating the design discharge.

(3) Non-uniform Flow. There are two types of steady state non-uniform flow:

- Gradually varied flow.

  Gradually varied flow is described as a steady state flow condition where the depth of water varies gradually over the length of the channel. Under this condition, the streamlines of flow are practically parallel and therefore, the assumption of hydrostatic pressure distribution is valid and uniform flow principles can be used to analyze the flow conditions.

- Rapidly varied flow.

  With the rapidly varied flow condition, there is a pronounced curvature of the flow streamlines and the assumption of hydrostatic pressure distribution is no longer valid, even for the continuous flow profile. A number of empirical procedures have been developed to address the various phenomena of rapidly varied flow. For additional discussion on the topic of rapidly varied flow, refer to "Open-Channel Hydraulics" by Chow.

866.3 Open Channel Flow Equations

The equations of open channel flow are based on uniform flow conditions. Some of these equations have been derived using basic conservation laws (e.g. conservation of energy) whereas others have been derived using an empirical approach.

(1) Continuity Equation. One of the fundamental concepts which must be satisfied in all flow problems is the continuity of flow. The continuity equation states that the mass of fluid per unit time passing every section in a stream of fluid is constant. The continuity equation may be expressed as follows:

$$Q = A_1V_1 = A_2V_2 = \ldots = A_nV_n$$

Where Q is the discharge, A is the cross-sectional flow area, and V is the mean flow velocity. This equation is not valid for spatially varied flow, i.e., where flow is entering or leaving along the length of channel under consideration.

(2) Bernoulli Equation. Water flowing in an open channel possesses two kinds of energy: (1) potential energy and (2) kinetic energy. Potential energy is due to the position of the water surface above some datum. Kinetic energy is due to the energy of the moving water. The total energy at a given section as expressed by the Bernoulli equation is equal to:

$$H = z + d + \frac{V^2}{2g}$$

where:

H = Total head, in feet of water

z = Distance above some datum, in feet

d = Depth of flow, in feet

$\frac{V^2}{2g}$ = Velocity head, in feet

g = Acceleration of gravity

= 32.2 feet per second squared

(3) Energy Equation. The basic principle used most often in hydraulic analysis is conservation of energy or the energy equation. For uniform flow conditions, the energy equation states that the energy at one section of a channel is equal to the energy at any downstream section plus the intervening energy losses. The energy equation, expressed in terms of the Bernoulli equation, is:

$$z_1 + d_1 + \frac{V_1^2}{2g} = z_2 + d_2 + \frac{V_2^2}{2g} + h_L$$

where:

h_L = Intervening head losses, in feet

(4) Manning's Equation. Several equations have been empirically derived for computing the average flow velocity within an open channel. One such equation is the Manning Equation. Assuming uniform and turbulent flow conditions, the mean flow velocity in an open channel can be computed as:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

where:

V = Mean velocity, in feet per second

n = Manning coefficient of roughness

S = Channel slope, in foot per feet

R = Hydraulic Radius, in feet

= A/WP

where:

A = Cross sectional flow area, in square feet

WP = Wetted perimeter, in feet

Commonly accepted values for Manning's roughness coefficient, n, based on materials and workmanship required in the Standard Specifications, are provided in Table 866.3A. The tabulated values take into account deterioration of the channel lining surface, distortion of the grade line due to unequal settlement, construction joints and normal surface irregularities. These average values should be modified to satisfy any foreseeable abnormal conditions. See Chapters 4 and 6 in HEC No. 15 for Manning's roughness equations for grass linings, RSP, cobble and gravel linings. Refer to Index 861.11 for a discussion of Manning's roughness coefficients for water quality channels.

Direct solutions for Manning's equation for many channels of trapezoidal, rectangular, triangular and circular cross sections can be found within the Channel Analysis subcomponent FHWA's Hydraulic Toolbox software program.

(5) *Conveyance Equation.* Often it is convenient to group the properties peculiar to the cross section into one term called the conveyance factor, K. The conveyance factor, as expressed by the Manning's equation, is equal to:

$$K = \frac{1.486}{n} AR^{2/3}$$

For the non-pressure, full flow condition, the geometric properties and conveyance of a channel section can be computed. Then for a given channel slope the discharge capacity can be easily determined.

(6) *Critical Flow.* A useful concept in hydraulic analysis is that of "specific energy". The specific energy at a given section is defined as the total energy, or total head, of the flowing water with respect to the channel bottom. For a channel of small slope;

$$E = d + \frac{V^2}{2g}$$

where:

E = Specific energy, in feet

Table 866.3A

Average Values for Manning's Roughness Coefficient (n)

Type of Channel	n value
Unlined Channels:	
Clay Loam	0.023
Sand	0.020
Gravel	0.030
Rock	0.040
Lined Channels:	
Portland Cement Concrete	0.014
Air Blown Mortar (troweled)	0.012
Air Blown Mortar (untroweled)	0.016
Air Blown Mortar (roughened)	0.025
Asphalt Concrete	0.016-0.018
Sacked Concrete	0.025
Pavement and Gutters:	
Portland Cement Concrete	0.013-0.015
Hot Mix Asphalt Concrete	0.016-0.018
Depressed Medians:	
Earth (without growth)	0.016 - 0.025
Earth (with growth)	0.050
Gravel (d_{50} = 1 in. flow depth $\leq$ 6 in.)	0.040
Gravel (d_{50} = 2 in. flow depth $\leq$ 6 in.)	0.056

NOTES:

For additional values of n, see HEC No. 15, Tables 2.1 and 2.2, and "Introduction to Highway Hydraulics", Hydraulic Design Series No. 4, FHWA Table 14.

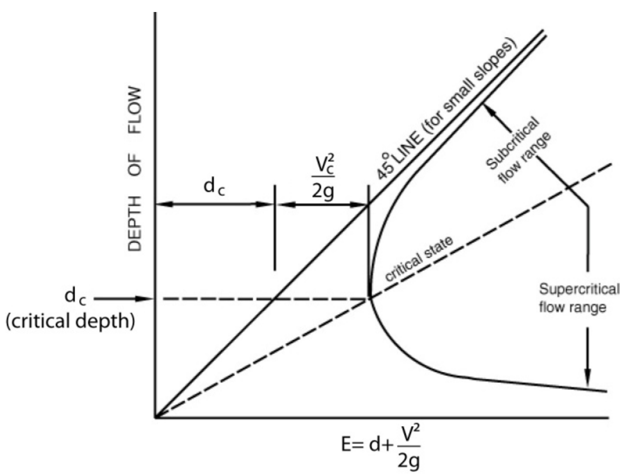
d = Depth of flow, in feet

$\frac{V^2}{2g}$ = Velocity head, in feet

When the depth of flow is plotted against the specific energy, for a given discharge and channel section, the resulting plot is called a specific energy diagram (see Figure 866.3C). The curve shows that for a given specific energy there are two possible depths, a high stage and a low stage. These flow depths are called alternate depths. Starting at the upper right of the curve with a large depth and small velocity, the specific energy decreases with a decrease in depth, reaching a minimum energy content at a depth of flow known as critical depth. A further decrease in flow depth results in a rapid increase in specific energy.

Flow at critical depth is called critical flow. The flow velocity at critical depth is called critical velocity. The channel slope which produces critical depth and critical velocity for a given discharge is the critical slope.

Figure 866.3C
Specific Energy Diagram



Uniform flow within approximately 10 percent of critical depth is unstable and should be avoided in design, if possible. The reason for this can be seen by referring to the specific energy diagram. As the flow approaches critical depth from either limb of the curve, a very small change in energy is required for the depth to abruptly change to the alternate depth on the opposite limb of the specific energy curve. If the unstable flow region cannot be avoided in design, the least favorable type of flow should be assumed for the design.

When the depth of flow is greater than critical depth, the velocity of flow is less than critical velocity for a given discharge and hence, the flow is subcritical. Conversely, when the depth of flow is less than critical depth, the flow is supercritical.

When velocities are supercritical, air entrainment may occur. This produces a bulking effect which increases the depth of flow. For concrete lined channels, the normal depth of flow with bulking can be computed by using a Manning's "n" value of 0.018 instead of the 0.014 value given in Table 866.3A. Air entrainment also causes a reduction in channel friction with a resulting increase in flow velocity. A Manning's "n" value of about 0.008 is recommended for computing the velocity and specific energy of flow in concrete-lined channels carrying supercritical flow

Critical depth is an important hydraulic parameter because it is always a hydraulic control. Hydraulic controls are points along the channel where the water level or depth of flow is limited to a predetermined level or can be computed directly from the quantity of flow. Flow must pass through critical depth in going from subcritical flow to supercritical flow. Typical locations of critical depth are at:

(a) Abrupt changes in channel slope when a flat (subcritical) slope is sharply increased to a steep (supercritical) slope,

(b) A channel constriction such as a culvert entrance under some conditions,

(c) The unsubmerged outlet of a culvert on subcritical slope, discharging into a wide channel or with a free fall at the outlet, and

(d) The crest of an overflow dam or weir.

Critical depth for a given channel is dependent on the channel geometry and discharge only, and is independent of channel slope and roughness.

When flow occurs at critical depth the following relationship must be satisfied

$$\frac{A^3}{T} = \frac{Q^2}{g}$$

where:

A = Cross sectional area, ft^2

T = Top width of water surface, ft

Q = Discharge, CFS

g = Acceleration of gravity, 32.2 ft/s^2

Critical depth formulas, based on the above equation, for various channel cross-sections include:

- Rectangular sections,

$$d_c = \left(\frac{q^2}{g}\right)^{1/3}$$

Where:

q = Flow per unit width, CFS

- Trapezoidal sections. The tables in King's "Handbook of Hydraulics" provide easy solutions for critical depth for channels of varying side slopes and bottom widths.
- Circular sections. The tables in King's "Handbook of Hydraulics" can be used for obtaining easy solutions for critical depth.

(7) *Froude Number.* The Froude number is a useful parameter which uniquely describes open flow. The Froude number is a dimensionless value:

$$Fr = \frac{V}{(gD)^{1/2}}$$

Where:

D = A/T = Hydraulic depth, in feet

Fr < 1.0 ==> Subcritical flow

Fr = 1.0 ==> Critical flow

Fr > 1.0 ==> Supercritical flow

866.4 Water Surface Profiles

Depending on the site conditions, accuracy required, and risks involved, a single section analysis may be sufficient to adequately describe the channel stage discharge relationship. The basic assumptions to a single section analysis are uniform cross section, slope, and Manning's "n" values which are generally applicable to most roadside and median channels. The condition of uniform flow in a channel at a known discharge is computed using the Manning's equation combined with the continuity equation:

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2}$$

The depth of uniform flow is solved by rearranging Manning's Equation to the form the given below. This equation is solved by trial and error by varying the depth of flow until the left side of the equation is zero:

$$\frac{Qn}{1.49S^{1/2}} - AR^{2/3} = 0$$

Per Index 866.3 (4), direct solutions for Manning's equation for many channels of trapezoidal, rectangular, triangular and circular cross sections can be found within the Channel Analysis subcomponent FHWA's Hydraulic Toolbox software program.

Where uniform flow conditions do not adequately describe the actual flow conditions (e.g., natural channels) or where additional accuracy is desired, the computation of complete water surface profiles for each discharge value may be necessary using detailed backwater analysis methods. Per Index 802.1(4)(g) contact the District Hydraulic Engineer for support.

Topic 867 - Channel Changes

867.1 General

Chapter 860 primarily addresses the design of small man-made open channels called roadside channels (gutters, ditches, swales etc.) that are constructed as part of a highway drainage system. However, both the terms ‘open channel’ or ‘channel’ may be applied to any natural or improved watercourse as well as roadside channels. See Index 861.1.

A channel change is any realignment or change in the hydraulic characteristics of an existing channel. Per Index 802.1(4)(g), contact the District Hydraulic Engineer for support.

The main reasons for channel changes to either natural or improved watercourses (flood control channels, irrigation channels etc.) within the right of way are to:

- Permit better drainage
- Improve flow conditions
- Protect the highway from flood damage
- Reduce right of way requirements

The guidelines in Topic 823 (Culvert Location) generally recommend alignment of the thalweg of the stream with the centerline of the culvert, however, for economic reasons, small skews should be eliminated, moderate skews retained and large skews reduced. Road crossings requiring fish passage are strongly encouraged to retain the natural alignment of the stream, regardless of the skew. Alignment of the culvert centerline with the channel approach angle aids debris passage during storm flows and minimizes hydraulic turbulence which may impede fish passage.

Sometimes a channel change may be to its vertical alignment. For example, inverted siphons or sag culverts may be used to carry irrigation channels crossing the right of way via vertical realignment entirely below the hydraulic grade line. However, maintenance concerns include sediment build-up and potential leakage problems with full-flow barrel(s). See Index 829.7(2) and Index 867.2 below.

867.2 Design Considerations

Channel changes should be designed with extreme caution and coordinated with District Hydraulics. Careful study of the channel characteristics upstream and downstream as well as within the channel change area is required to achieve a safe and effective design.

Channel changes may result in a decreased surface roughness or increased channel slope. As a result the following may occur:

- Higher velocities which result in damage due to scour
- Sedimentation and meandering at downstream end of channel change
- A flattened downstream gradient which progresses upstream undercutting the channel banks or highway fill
- Flattened downstream gradient or channel restrictions may create undesirable backwater conditions.

A channel change perched above the bottom of an old flood stage stream bed may cause the stream to return to its old channel during a subsequent flood. In addition, the designer should consult with Geotechnical Services to ensure that infiltration through the bank would not be problematic.

Topic 868 - Freeboard Considerations

868.1 General

Freeboard is the extra height of bank above the design depth where overflow is predicted to cause damage. Freeboard allowances will vary with each situation.

866.2 Height of Freeboard

(1) *Straight Alignment*. In channels where overflow may cause substantial damage, a guide for freeboard height for channels on a straight alignment, is provided in Table 868.2

Table 868.2

Guide to Freeboard Height

Shape of Channel	Subcritical Flow	Supercritical Flow
Rectangular	0.1 He	0.20 d
Trapezoidal	0.2 He	0.25 d

where:

He = Energy head, in feet

d = Depth of flow, in feet for a straight alignment

(2) *Critical Flow*. An unstable zone of flow occurs where the flow is near critical state. This is characterized by random waves. An allowance for waves should be added to the normal depth when the slope of the channel is between 0.7 S_c and 1.3 S_c.

$$H_w = 0.25 d_c \left[1 - 11.1 \left(\frac{S}{S_c} - 1 \right)^2 \right]$$

where:

H_w = height of wave, in feet

d_c = critical depth, in feet

S = slope of channel, in foot per feet

S_c = critical slope, in foot per feet

(3) *Superelevation.* The height of freeboard discussed above does not provide for superelevation of the water surface on curved alignments.

Flow around a curve will result in a rise of the water surface on the outside of the curve and extra lining is necessary to guard against overtopping.

Additional freeboard is necessary in bends and can be calculated use the following equation:

$$\Delta d = \frac{V^2 T}{g R_c}$$

where:

Δd = Additional freeboard required because of superelevation, feet

V = Average channel veloctiy, ft/s

T = Water surface top width, ft

G = Acceleration due to gravity, ft/s^2

R_c = Radius of curvature of the bend to the channel centerline, ft

See HEC No. 15, Chapter 3, for shear stress considerations around bends.

CHAPTER 870 CHANNEL AND SHORE PROTECTION - EROSION CONTROL

Topic 871 - General

Index 871.1 - Introduction

Highways, bikeways, pedestrian facilities and appurtenant installations are often attracted to parallel locations along streams, coastal zones and lake shores. These locations are under attack from the action of waves and flowing water, and may require protective measures.

Channel and shore protection can be a major element in the design, construction, and maintenance of highways. This section deals with procedures, methods, devices, and materials commonly used to mitigate the damaging effects of flowing water and wave action on transportation facilities and adjacent properties. Potential sites for such measures should be reviewed in conjunction with other features of the project such as long and short term protection of downstream water quality, aesthetic compatibility with surrounding environment, and ability of the newly created ecological system to survive with minimal maintenance. See Index 110.2 for further information on water quality and environmental concerns related to erosion control.

Refer to Topic 870 for definitions of drainage terms.

871.2 Design Philosophy

In each district there should be a designer or advisor, usually the District Hydraulic Engineer, knowledgeable in the application of bank protection principles and the performance of existing works. Information is also available from headquarters specialists in the Division of Design and Structures Design in the Division of Engineering Services (DES). The most effective designs result from involvement with Design, Landscape Architecture, Structures, Construction, and Maintenance (for further discussion on functional responsibilities see Topic 802).

There are a number of ways to deal with the problem of wave action and stream flow.

- The simplest way and generally the surest of success and permanence, is to locate the facility away from the erosive forces. This is not always feasible or economical, but should be the first consideration. Locating the facility to higher ground or solid support should never be overlooked, even when it requires excavation of solid rock, since excavated rock may serve as a valuable material for protection at other points of attack.
- The most commonly used method is to armor the embankment with a more resistant material like rock slope protection. The type of material to be used for the protection is discussed under Topic 872.
- A third method is to reduce the force of the attacking water. This is often done by means of retards, permeable jetties and various plantings such as willows. Plantings once established not only reduce stream velocity near the bank during heavy flows, but their roots add structure to the bank material.
- Another method is to direct the attacking water away from the embankment. In the case of wave attack, additional beach may be created between the embankment and the water by means of groins and sills which trap littoral drift or hold imported sand. In the case of stream attack, a new channel can be created or the stream can be diverted away from the embankment by the use of jetties, baffles, deflectors, groins or spurs.

Combinations of the above four methods may be used. Even protective works destroyed in floods have proven to be effective and cost efficient in minimizing damage to transportation facilities.

Design of protective features should be governed by the importance of the facility and appropriate design principles. Some of the factors which should be considered are:

- *Roughness*. Revetments generally are less resistant to flow than the natural channel bank. Channel roughness can be significantly reduced if a rocky vegetated bank is denuded of trees and rock outcrops. When a rough natural bank is replaced by a smooth revetment, the current is accelerated, increasing its power to erode, especially along the toe and downstream end of the revetment. Except in narrowed channels, protective elements should approximate natural roughness. Retards, baffles and jetties can simulate the effect of trees and boulders along natural banks and in overflow channels.

- *Undercutting*. Particular attention must be paid to protecting the toe of revetments against undercutting caused by the accelerated current along smoothed banks, since this is the most common cause of bank failure.
- *Standardization*. Standardization should be a guide but not a restriction in designing the elements and connections of protective structures.
- *Expendability*. The primary objective of the design is the security of the transportation facility, not security of the protective structure. Less costly replaceable protection may be more economical than expensive permanent structures.
- *Dependability*. An expensive structure is warranted primarily where transportation facilities carry high traffic volumes, where no reasonable detour is available, or where facility replacement is very expensive.
- *Longevity*. Short-lived structures or materials may be economical for temporary situations. Expensive revetments should not be placed on banks likely to be buried in widened embankments, nor on banks attacked by transient meander of mature streams.
- *Materials*. Optimum use should be made of local materials, considering the cost of special handling. Specific gravity of stone is a major factor in shore protection and the specified minimum should not be lowered without increasing the mass of stones. For example, 10 percent decrease in specific gravity requires a 55 percent increase in mass (say from a 9 ton stone to a 14 ton stone) for equivalent protection.
- *Selection*. Selection of class and type of protection should be guided by the intended function of the installation.
- *Limits*. Horizontal and vertical limits of protection should be carefully designed. The bottom limit should be secure against toe scour. The top limit should not arbitrarily be at high-water mark, but above it if overtopping would cause excessive damage and below it if floods move slowly along the upper bank. The end limits should reach and conform to durable natural features or be secure with respect to design parameters.

871.3 Selected References

Hydraulic and drainage related publications are listed by source under Topic 807. References specifically related to slope protection measures are repeated here for convenience.

(a) FHWA Hydraulic Engineering Circulars (HEC) -- The following five circulars were developed to assist the designer in using various types of slope protection and channel linings:

- HEC 11, Design of Riprap Revetment (2000)
- HEC 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (2006)
- HEC 15, Design of Roadside Channels with Flexible Linings (2005).
- HEC 18, Evaluating Scour at Bridges (2001)
- HEC 20, Stream Stability at Highway Structures (2001)
- HEC 23, Bridge Scour and Stream Instability Countermeasures (2009)
- HEC 25, Highways in the Coastal Environment (2008)

(b) FHWA Hydraulic Design Series (HDS) No. 6, River Engineering for Highway Encroachments (2001) -- A comprehensive treatise of natural and man-made impacts and responses on the river environment, sediment transport, bed and bank stabilization, and countermeasures.

(c) AASHTO Highway Drainage Guidelines -- General guidelines for good erosion control practices are covered in Volume III - Erosion and Sediment Control in Highway Construction, and Volume XI - Guidelines for Highways Along Coastal Zones and Lakeshores.

(d) AASHTO Drainage Manual (MDM) (2003) – Refer to Chapters; 11 – Energy Dissipators; 16 – Erosion and Sediment Control; 17 – Bank Protection; and 18 – Coastal Zone. The MDM provides guidance on engineering practice in conformance with FHWA's HEC and HDS publications and other nationally recognized engineering policy and procedural documents.

(e) U.S. Army Corps of Engineers Manuals. The following manuals are used throughout the U.S. as a primary resource for the design and analysis of coastal features:

- Shore Protection Manual (SPM) (1984) – Comprehensive two volume guidance on wave and shore processes and methods for shore protection. No longer in publication but still referenced pending completion of the Coastal Engineering Manual.
- Design of Coastal Revetments, Seawalls, and Bulkheads. Engineering Manual 1110-2-1614 (1995) – Supersedes portions of Volume 2 of the Shore Protection Manual (SPM).
- Coastal Engineering Manual. Engineer Manual (EM) 1110-2-1100 (2002) – Published in six parts plus an appendix, this set of documents, once complete, will supersede the SPM and EM 1110-2-1614. As of this writing Parts I thru V and the appendix are completed and available. Parts V and VI are considered "Engineering – Based" and present information on design process and selection of appropriate types of solutions to various coastal challenges.

Topic 872 - Planning and Location Studies

872.1 Planning

The development of cost effective protective works requires careful planning. Planning begins with site investigation. The selection of the class of protection can be determined during or following site investigation. For some sites the choice is obvious; at other sites several alternatives or combinations may be applicable. See the FHWA's HDS No. 6, River Engineering for Highway Encroachments for a complete and thorough discussion of hydraulic and environmental design considerations associated with hydraulic structures in moveable boundary waterways.

Some specific site conditions that may dictate selection of a class and type of protection different from those shown in Table 872.1 are:

- Available right of way.
- Available materials.
- Possible damage to other properties through streamflow diversion or increased velocity.
- Environmental concerns.
- Channel capacity or conveyance.
- Conformance to new or existing structures.
- Provisions for side drainage, either surface waters or intersecting streams or rivers.

The first step is to determine the limits of the protection with respect to length, depth and the degree of security required.

Considerations at this stage are:

- The severity of attack.
- The present alignment of the stream or river and potential meander changes.
- The ratio of cost of highway replacement versus cost of protection.
- Whether the protection need be permanent or temporary.
- Analysis of foundation and materials explorations.

The second step is the selection and layout of protective elements in relation to the highway facility.

872.2 Class and Type of Protection

Protective devices are classified according to their function. They are further categorized as to the type of material from which they are constructed or shape of the device. For additional information on specific material types and shapes see Topic 873, Design Concepts.

There are two basic classes of protection, armor treatment and training works. Table 872.1 relates different location environments to these classes of protection.

872.3 Site Consideration

The determination of the lengths, heights, alignment, and positioning of the protection are affected to a large extent by the facility location environment.

An evaluation is required for any proposed highway construction or improvement that encroaches on a floodplain. See Topic 804, Floodplain Encroachments for detailed procedures and guidelines.

(1) *Young Valley.* Typically young valleys are narrow V-shaped valleys with streams on steep gradients. At flood stage, the stream flow covers all or most of the valley floor. The usual situation for such locations is a structure crossing a well-defined channel in which the design discharge will flow at a moderate to high velocity.

   (a) Cross-Channel Location. A cross channel location is a highway crossing a stream on normal or skewed alignment. The erosive forces of parallel flow associated with a normal crossing are generally less of a threat than the impinging and eddy flows associated with a skewed crossing. The effect of constriction by projection of the roadway embankment into the channel should be assessed.

   Characteristics to be considered include:

   - Stream velocity.
   - Scouring action of stream.
   - Bank stability.
   - Channel constrictions (artificial or natural).
   - Nature of flow (tangential or curvilinear).
   - Areas of impingement at various stages.
   - Security of leading and trailing edges.

   Common protection failures occur from:

   - Undermining of the toe (inadequate depth/size of foundation), see Figure 872.1 and Table 872.2.
   - Local erosion due to eddy currents.
   - Inadequate upstream and downstream terminals or transitions to erosion-resistant banks or outcrops.
   - Structural inadequacy at points of impingement overtopping.
   - Inadequate rock size, see Table 872.2.
   - Lack of proper gradation/ layering/ RSP fabric, leading to loss of embankment, see Table 872.2.

   Any of the more substantial armor treatments can function properly in such exposures providing precautions are taken to alleviate the probable causes of failure. If the foundation is questionable for concreted-rock or other rigid types it would not be necessary to reject them from consideration but only to provide a more acceptable treatment of the foundation, such as heavy rock or sheet piling.

Figure 872.1

Slope Failure Due to Loss of Toe

Table 872.1

Guide to Selection of Protection

Location	Armor										Training										
	Flexible					Rigid					Guide Dikes Retards & Jetties				Groins				Baffles		
			Mattresses																		
	Vegetation	Riprap	Gabions	Conc. Blocks	Fabric Filled	Grouted Rock	Stacked Conc.	Conc. Lined	Cribs	Bulk heads	Earth	Fencing	Piling	Other	Rock	Grouted Rock	Piling	Other	Drop Structure	Fencing	Rock Earth
Cross Channel																					
Young Valley		X	Ø			X			X	X											
Mature Valley		X	Ø	Ø	Ø	X			X	X	X	X	X	X	X	X			X	X	X
Parallel Encroachment																					
Young Valley		X	Ø			X			X	X											
Mature Valley	X	X	Ø	Ø	Ø	X	Ø		X	X	X	X	X	X	X	X	X	X	X	X	X
Lakes and Tidals Basins	X	X	Ø	Ø	Ø	X	Ø			X											
Ocean Front		X	Ø	Ø						X					X	X	X	X			
Desert-wash																					
Top debris cone		X	Ø	Ø	Ø	X				X											
Center debris cone		X	Ø	Ø	Ø	X													X	X	X
Bottomdebris cone		X	Ø	Ø	Ø	X													X	X	X
Overflow and floodplain	X	X	Ø			X	Ø				X	X	X	X							
Artificial channel	X	X	Ø	Ø	Ø	X	Ø	X													
Culvert																					
Inlet		X				X	Ø			X											
Outlet		X				X	Ø			X											
Bridge																					
Abutment		X		Ø	Ø	X	Ø	X													
Upstream		X				X					X	X	X	X							
Downstream		X				X					X	X	X	X					X		
Roadside ditch	X	X				X	Ø	X													

Ø Where large rock for riprap is not available

Table 872.2

Failure Modes and Effects Analysis for Riprap Revetment

Failure Modes	Effects on Other Components	Effects on Whole System	Detection Methods	Compensating Provisions
Translational slope or slump (slope failure)	Disruption of armor layer	Catastrophic failure	• Mound of rock at bank toe • Unprotected upper bank	• Reduce bank slope • Use more angular or smaller rock • Use granular filter rather than geotextile fabric
Particle erosion (rock undersized)	Loss of armor layer, erosion of filter	Progressive failure	• Rock moved downstream from original location • Exposure of filter	• Increase rock size • Modify rock gradation
Piping or erosion beneath armor (improper filter)	Displacement of armor layer	Progressive failure	• Scalloping of upper bank • Bank cutting • Void beneath and between rocks	• Use appropriate granular or geotextile filter
Loss of toe or key (under designed)	Displacement or disruption or armor layer	Catastrophic failure	• Slumping of rock • Unprotected upper bank	• Increase size, thickness, depth or extent of toe or key

Whether the highway crosses a stream channel on a bridge or over a culvert, economic considerations often lead to constriction of the waterway. The most common constriction is in width, to shorten the structure. Next in frequency is obstruction by piers and bents of bridges or partitions of multiple culverts.

The risk of constricting the width of the waterway is closely related to the relative conveyance of the natural waterway obstructed, the channel scour, and to the channel migration. Constricting the width of flow at structures has the following effects:

- Increase in the upstream water surface elevation (backwater profile).
- Increase in flow velocity through the structure opening (waterway).
- Causes eddy currents around the upstream and downstream ends of the structure.

Unless protection is provided the eddy currents can erode the approach roadway embankment and the accelerated flow can cause scour at bridge abutments. The effects of erosion can be reduced by providing transitions from natural to constricted and back to natural sections, either by relatively short wingwalls or by relatively long training embankments or structures.

Channel changes, if properly designed, can improve conditions of a crossing by reducing skew and curvature and enlarging the main channel. Unfortunately there are "side effects" which actually increase erosion potential. Velocity is almost always increased by the channel change, both by a reduction of channel roughness and increase of slope due to channel shortening. In addition, channel changes affecting stream gradient may have upstream and/or downstream effects as the stream adjusts in relation to its sediment load.

At crossing locations, lateral erosion can be controlled by positive protection, such as armor on the banks, rock spurs to deflect currents away from the banks, retards to reduce riparian velocity, or vertical walls or bulkheads. The life cycle cost of such devices should be considered in the economic studies to choose a bridge length which minimizes total cost.

Accurate estimates of anticipated scour depths are a prerequisite for safe, cost effective designs. Design criteria require that bridge foundations be placed below anticipated scour depths. For this reason the design of protection to control scour at such locations is seldom necessary for new construction. However, if scour may undercut the toes of dikes or embankments positive methods including self-adjusting armor at the toe, jetties or retards to divert scouring currents away from the toe, or sill-shaped baffles interrupting transport of bedloads should be considered.

There is the potential for instability from saturated or inundated embankments at crossings with embankments projecting into the channel. Failures are usually reported as "washouts", but several distinct processes should be noted:

- Saturation of an embankment reduces its angle of repose. Granular fills with high permeability may "dissolve" steadily or slough progressively. Cohesive fills are less permeable, but failures have occurred during falling stages.
- As eddies carve scallops in the embankment, saturation can be accelerated and complete failure may be rapid. Partial or total losses can occur due to an upstream eddy, a downstream eddy, or both eddies eroding toward a central conjunction. Training devices or armor can be employed to prevent damage.
- If the fill is pervious and the pavement overtopped, the buoyant pressure under the slab will exceed the weight of slab and shallow overflow by the pressure head of the hydraulic drop at the

shoulder line. A flat slab of thickness, t, will float when the upstream stage is 4t higher than the top of the slab. Thereafter the saturated fill usually fails rapidly by a combination of erosion and sloughing. This problem can occur or be increased when curbs, dikes, or emergency sandbags maintain a differential stage at the embankment shoulder. It is increased by an impervious or less pervious mass within the fill. Control of flotation, insofar as bank protection is concerned, should be obtained by using impervious armor on the upstream face of the embankment and a pervious armor on the downstream face.

Culvert problem locations generally occur in and along the downstream transition. Sharp divergence of the high velocity flow develops outward components of velocity which attack the banks directly by impingement and indirectly by eddies entrained in quieter water. Downward components and the high velocity near the bed cause the scour at the end of the apron.

Standard plans of warped wingwalls have been developed for a smooth transition from the culvert to a trapezoidal channel section. A rough revetment extension to the concrete wingwalls is often necessary to reduce high velocity to approximate natural flow. Energy dissipaters may be used to shorten the deceleration process when such a transition would be too long to be economical. Bank protection at the end of wingwalls is more cost effective in most cases.

(b) Parallel Location. With parallel locations the risk of erosion damage along young streams increases where valleys narrow and gradients steepen. The risk of erosion damage is greatest along the outer bend of natural meanders or where highway embankment encroaches on the main channel.

The *encroaching* parallel location is very common, especially for highways following mountain streams in narrow young valleys or canyons. Much of the roadway is supported on top of the bank or a berm and the outer embankment encroaches on the channel in a zone of low to moderate velocity. Channel banks are generally stable and protection, except at points of impingement, is seldom necessary.

The *constricting* parallel location is an extreme case of encroaching location, causing such impairment of channel that acceleration of the stream through the constriction increases its attack on the highway embankment requiring extra protection, or additional waterway must be provided by deepening or widening along the far bank of the stream.

In young valleys, streams are capable of high velocity flows during flood stages that may be damaging to adjacent highway facilities. Locating the highway to higher ground or solid support is always the preferred alternative when practical.

Characteristics to be considered include:

- High velocity flow.
- Narrow confined channels.
- Accentuated impingement.
- Swift overflow.
- Disturbed flow due to rock outcrops on the banks or within the main channel.
- Alterations in flow patterns due to the entrance of side streams into the main channel.

Protective methods that have proven effective are:

- Rock slope protection.
- Concreted-rock slope protection.
- Walls of masonry and concrete.
- Articulated concrete block revetments.
- Sacked concrete.
- Cribs walls of various materials.

(2) Mature Valley. Typically mature valleys are broad V-shaped valleys with associated

floodplains. The gradient and velocity of the stream are low to moderate. In addition to the general information previously given, the following applies to mature valleys.

(a) Cross-Channel Location. The usual situation is a structure crossing a braided or meandering normal flow channel. The marginal area subject to overflow is usually traversed by the highway on a raised embankment and may have long approaches extending from both banks.

Characteristics to be considered include:

- Shifting of the main channel.
- Skew of the stream to the structure.
- Foundation in deep alluvium.
- Erodible embankment materials.
- Channel constrictions, either artificial or natural, which may affect or control the future course of the stream.
- Variable flow characteristics at various stages.
- Stream acceleration at the structure.

Armor protection has proven effective to prevent erosion of road approach embankments, supplemented if necessary by stream training devices such as guide dikes, permeable retards or jetties to direct the stream through the structure. The abutments should not depend on the training dikes to protect them from erosion and scour. At bridge ends one of the more substantial armor types may be required, but bridge approach embankments affected only by overflow seldom require more than a light revetment, such as a thin layer of rocky material, vegetation, or a fencing along the toe of slope. For channel flow control upstream, the size and type of training system ranges from pile wings for high velocity, through permeable jetties for moderate velocity, to the earth dike suitable for low velocity.

The more common failures in this situation occur from:

- Lack of upstream control of channel alignment.
- Damage of unprotected embankments by overflow and return flow.
- Undercut foundations.
- Formation of eddies at abrupt changes in channel.
- Stranding of drift in the converging channel.

(b) Parallel Location. Parallel highways along mature rivers are often situated on or behind levees built, protected and maintained by other agencies. Along other streams, rather extensive protective measures may be required to control the action of these meandering streams.

Channel change is an important factor in locations parallel to mature streams. The channel change may be to close an embayment, to cut off an oxbow, or to shift the alignment of a long reach of a stream. In any case, positive means must be adopted to prevent the return of the stream to its natural course. For a straight channel, the upstream end is critical, usually requiring bank protection equivalent to the facing of a dam. On a curved channel change, all of the outer bend may be critical, requiring continuous protection. For a channel much shorter than the natural channel, particularly for elimination of an oxbow, the corresponding increase in gradient may require transverse weirs as grade control structures to prevent undercutting. For unusual channel changes, preliminary plans and hydraulic data must be submitted to FHWA for approval (see Index 805.5).

(3) Lakes and Tidal Basins. Highways adjacent to lakes or basins may be at risk from wave generated erosion. All bodies of waters generate waves. Height of waves is a function of fetch and depth. Erosion along embankments behind shallow coves is reduced because the higher waves break upon reaching a shoal in shallow water. The threat of erosion in deep water at headlands or along causeways is increased. Constant exposure to even the

rippling of tiny waves may cause severe erosion of some soils.

Older lakes normally have thick beds of precipitated silt and organic matter. Bank protection along or across such lakes must be designed to suit the available foundation. It is usually more practical to use lightweight or self-adjusting armor types supported by the soft bed materials than to excavate the mud to stiffer underlying soils.

In fresh waters, effective protection can often be provided by the establishment of vegetation, but planners should not overlook the possibility of moderate erosion before the vegetative cover becomes established. A light armor treatment should be adequate for this transitional period.

(4) *Ocean Front Locations.* Wave action is the erosive force affecting the reliability of highway locations along the coast. The corrosive effect of salt water is also a major concern for hydraulic structures located along the coastline. Headlands and rocks that have historically withstood the relentless pounding of tide and waves can usually be relied on to continue to protect adjacent highway locations founded upon them. The need for shore protection structures is, therefore, generally limited to highway locations along the top or bottom of bluffs having a history of sloughing and along beach fronts.

Beach protection considerations include:

- Attack by waves.
- Littoral drift of the beach sands.
- Seasonal shifts of the shore.
- Foundation for protective structures.

Wave attack on a beach is less severe than on a headland, due to the gradual shoaling of the bed which trips incoming waves into a series of breakers called a surf.

Littoral drift of beach sands may either be an asset or a liability. If sand is plentiful, a new beach will be built in front of the highway embankment, reducing the depth of water at its toe and the corresponding height of the waves attacking it. If sand supply is less plentiful or subject to seasonal variations, the new beach can be induced or retained by groins.

If sand is in scant supply, backwash from a revetment tends to degrade the beach or bed even more than the seasonal variation, and an allowance should be made for this scour when designing the revetment, both as to weight of stones and depth of foundation. Groins may be ineffective for such locations; if they succeeded in trapping some littoral drift, downcoast beaches would recede from undernourishment.

Seasonal shifts of the shore line result from combinations of:

- Ranges of tide.
- Reversal of littoral currents.
- Changed direction of prevailing onshore winds.
- Attack by swell.

Generally the shift is a recession, increasing the exposure of beach locations to the hazard of damage by wave action. On strands or along extensive embayments, recession at one end may result in deposition at the other. Observations made during location assessment should include investigation of this phenomenon. For strands, the hazard may be avoided by locating the highway on the backshore facing the lagoon.

Foundation conditions vary widely for beach locations. On a receding shore, good bearing may be found on soft but substantial rock underlying a thin mantle of sand. Bed stones and even gravity walls have been founded successfully on such foundations. Spits and strands, however, are radically different, often with softer clays or organic materials underlying the sand. Sand is usually plentiful at such locations, subsidence is a greater hazard than scour, and location should anticipate a "floating" foundation for flexible, self-adjusting types of protection.

In planning ocean-front locations, the primary decision is a choice of (1) alignment far enough inshore to avoid wave attack, (2) armor on the embankment face, or (3) off shore devices like groins to aggrade the beach at embankment toe.

See Index 873.3(2) for further discussion on determining the size of rocks necessary in shore protection for various wave heights.

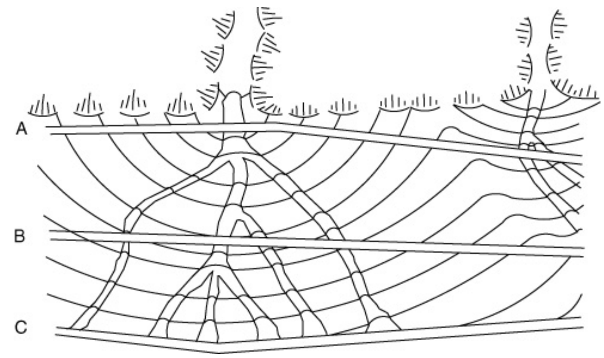
(5) *Desert Wash Locations.* Special consideration should be given to highway locations across the natural geographical features of desert washes, sand dunes, and other similar regions.

Desert washes are a prominent feature of the physiography of California. Many long stretches of highway are located across a succession of outwash cones. Infrequent discharge is typically wide and shallow, transporting large volumes of solids, both mineral and organic. Rather than bridge the natural channels, the generally accepted technique is to concentrate the flow by a series of guide dikes leading like a funnel to a relatively short crossing.

An important consideration at these locations is instability of the channel, see Figure 872.2. For a location at the top of a cone (Line A), discharge is maximum, but the single channel emerging from the uplands is usually stable. For a location at the bottom of the cone (Line C), instability is maximum with poor definition of the channel, but discharge is reduced by infiltration and stream dispersion. The energy of the stream is usually dissipated so that any protection required is minimal. The least desirable location is midway between top and bottom (Line B), where large discharge may approach the highway in any of several old channels or break out on a new line. Control may require dikes continuously from the top of the cone to such a mid-cone site with slope protection added near the highway where the converging flow is accelerated. See Figure 872.3, which depicts a typical alluvial fan.

Also common are roadway alignments which longitudinally encroach, or are fully within the desert wash floodplain, see Figure 872.4. Re-alignment to a stable location should be the first consideration, but restrictions imposed by federal or state agencies (National Park Service, USDA Forest Service, etc.) may preclude that option, somewhat similar to transverse crossings. The designer may need to consider allowing frequent overtopping and increased sediment removal maintenance since an "all weather design" within these regimes can often lead to large scale roadway washout.

Figure 872.2
Alternative Highway Locations Across Debris Cone

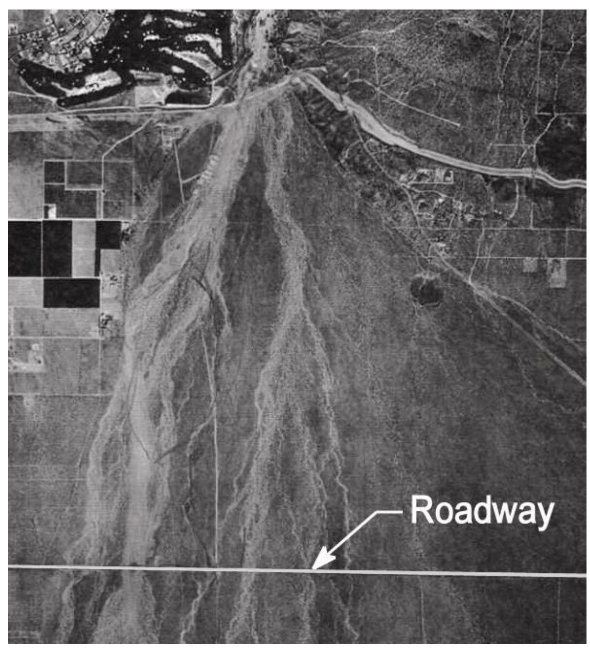


A. Cross at a single definite channel

B. A series of unstable indefinite channels and

C. A widely dispersed and diminished flow

Figure 872.3
Alluvial Fan



Typical multi-channel stream threads on alluvial fan. Note location of roadway crossing unstable channels.

Figure 872.4
Desert Wash Longitudinal Encroachment

Road washout due to longitudinal location in desert wash channel

Characteristics to be considered include:

- The intensity of rainfall and subsequent run-off.
- The relatively large volumes of solids that are carried in such run-off.
- The lack of definition and permanence of the channel.
- The scour depths that can be anticipated.
- The lack of good foundation.

Effective protective methods include armor along the highway and at structures and the probable need for baffles to control the direction and velocity of flow. Installations of rock, fence, palisades, slope paving, and dikes have been successful.

The Federal Emergency Management Agency (FEMA) Flood Hazard Mapping website contains information on recognizing alluvial fan landforms and methods for defining active and inactive areas. See their "Guidelines for Determining Flood Hazards on Alluvial Fans" at http://www.fema.gov/fhm/ft_tocs.shtm.

872.4 Data Needs

The types and amount of data needed for planning and analysis of bank and shore protection varies from project to project depending upon the class and extent of the proposed protection, site location environment, and geographic area. The data that is collected and developed including preliminary calculations, and alternatives considered should be documented in project development reports (Environmental Document, Project Report, etc.) or as a minimum in the project file. These records serve to guide the detailed designs, and provide reference background for analysis of environmental impacts and other needs such as permit applications and historical documentation for any litigation which may arise.

Recommendations for data needs can be requested from the District Hydraulics Engineer or determined from Chapter 8 of FHWA's HDS No. 6, for a more complete discussion of data needs for highway crossings and encroachments on rivers. Further references to data needs are contained in Chapter 810, Hydrology and FHWA's HDS No. 2, Highway Hydrology.

Topic 873 - Design Concepts

873.1 Introduction

No attempt will be made here to describe in detail all of the various devices that have been used to protect embankments against scour. Methods and devices not described may be used when justified by economical analysis. Not all publicized treatments are necessarily suited to existing conditions for a specific project.

A set of plans and specifications must be prepared to define and describe the protection that the design engineer has in mind. These plans should show controlling factors and an end product in such detail that there will be no dispute between the construction engineer and contractor. To serve the dual objectives of adequacy and economy, plans and specifications should be precise in defining materials to be incorporated in the work, and flexible in describing methods of construction or conformance of the end product to working lines and grades.

Recommendations on channel lining, slope protection, and erosion control materials can be requested from the District Hydraulic Engineer, the District Materials Branch and the Office of State Highway Drainage Design in Headquarters. The District Landscape Architect will provide recommendations for temporary and permanent erosion and sediment control measures. The

Caltrans Bank and Shore Protection Committee is available on request to provide expert advice on extraordinary situations or problems and to provide evaluation and formal approvals for acceptable non-standard designs. See Index 802.3 for further information on the organization and functions of the Committee.

Combinations of armor-type protection can be used, the slope revetment being of one type and the foundation treatment of another. The use of rigid, non-flexible slope revetment may require a flexible, self-adjusting foundation for example: concreted-rock on the slope with heavy rock foundation below, or PCC slope paving with a steel sheet-pile cutoff wall for foundation.

Bank protection may be damaged while serving its primary purpose. Lower cost replaceable facilities may be more economical than expensive permanent structures. However, an expensive structure may be economically warranted for highways carrying large volumes of traffic or for which no detour is available.

Cost of stone is extremely sensitive to location. Variables are length of haul, efficiency of the quarry in producing acceptable sizes, royalty to quarry and, necessity for stockpiling and rehandling. On some projects the stone may be available in roadway excavation.

873.2 Design High Water and Hydraulics

The most important, and often the most perplexing obligation, in the design of bank and shore protection features is the determination of the appropriate design high water elevation to be used. The design flood stage elevation should be chosen that best satisfies site conditions and level of risk associated with the encroachment. The basis for determining the design frequency, velocity, backwater, and other limiting factors should include an evaluation of the consequences of failure on the highway facility and adjacent property. Stream stability and sediment transport of a watercourse are critical factors in the evaluation process that should be carefully weighted and documented. Designs should not be based on an arbitrary storm or flood frequency.

A suggested starting point of reference for the determination of the design high water level is that the protection withstands high water levels caused by meteorological conditions having a recurrence interval of one-half the service life of the protected facility. For example, a modern highway embankment can reasonably be expected to have a service life of 100 years or more. It would therefore be appropriate to base the preliminary evaluation on a high water elevation resulting from a storm or flood with a 2 percent probability of exceedance (50 year frequency of recurrence). The first evaluation may have to be adjusted, either up or down, to conform with a subsequent analysis which considers the importance of the encroachment and level of related risks.

There is always some risk associated with the design of protection features. Special attention must be given to life threatening risks such as those associated with floodplain encroachments. Significant floodplain risks are classified as those having probability of:

- Catastrophic failure with loss of life.
- Disruption of fire and ambulance services or closing of the only evacuation route available to a community.

Refer to Topic 804, Floodplain Encroachments, for further discussion on evaluation of risks and impacts.

(1) *Streambank Locations.* The velocity along the banks of watercourses with smooth or uniformly rough tangent reaches may only be a small percentage of the average stream velocity. However, local irregularities of the bank and streambed may cause turbulence that can result in the bank velocity being greater than that of the central thread of the stream. The location of these irregularities is not always permanent as they may be caused by local scour, deposition of rock and sand, or stranding of drift during high water changes. It is rarely economical to protect against all possibilities and therefore some damage should always be anticipated during high water stages.

Essential to the design of streambank protection is sufficient information on the characteristics of the watercourse under consideration. For proper analysis, information on the following types of watercourse characteristics must be developed or obtained:

- Design Discharge
- Design High Water Level
- Flow Types
- Channel Geometry
- Flow Resistance
- Sediment Transport

Refer to Chapter 810, Hydrology, for a general discussion on hydrologic analysis and specifically to Topic 817, Flood Magnitudes; Topic 818, Flood Probability and Frequency; and Topic 819, Estimating Design Discharge. For a detailed discussion on the fundamentals of alluvial channel flow, refer to Chapter 3, HDS No. 6, and to Chapter 4, HDS No. 6, for further information on sediment transport.

(2) Ocean & Lake Shore Locations. Information needed to design shore protection is:

- Design High Water Level
- Design Wave Height

(a) Design High Water Level. The flood stage elevation on a lake or reservoir is usually the result of inflow from upland runoff. If the water stored in a reservoir is used for power generation, flood control, or irrigation, the design high water elevation should be based on the owners schedule of operation.

Except for inland tidal basins affected by wind tides, floods and seiches, the static or still-water level used for design of shore protection is the highest tide. In tide tables, this is the stage of the highest tide above "tide-table datum" at MLLW. To convert this to MSL datum there must be subtracted a datum equation (2.5 feet to 3.9 feet) factor. If datum differs from MSL datum, a further correction is necessary. These steps should be undertaken with care and independently checked. Common errors are:

- Ignoring the datum equation.
- Adding the factor instead of subtracting it.
- Using half the diurnal range as the stage of high water.

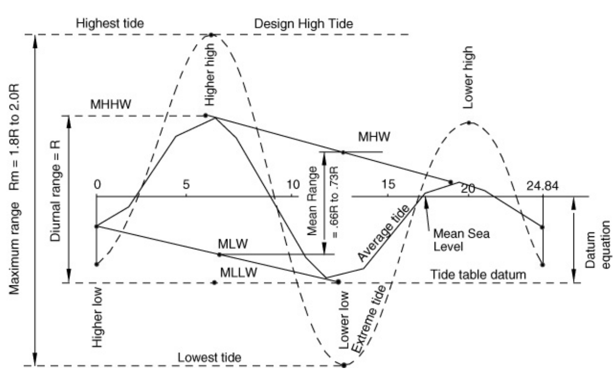
To clarify the determination of design high-water, Fig. 873.2A shows the *Highest Tide* in its relation to an extreme-tide cycle and to a hypothetical average-tide cycle, together with nomenclature pertinent to three definitions of tidal range. Note that the cycles have two highs and two lows. The average of all the higher highs for a long period (preferably in multiples of the 19-yr. metonic cycle) is MHHW, and of all the *lower* lows, MLLW. The vertical difference between them is the *diurnal range*.

Particularly on the Pacific coast where MLLW is datum for tide tables, the stage of MHHW is numerically equal to diurnal range.

The average of all highs (indicated graphically as the mean of higher high and lower high) is the MHW, and of all the lows, MLW. Vertical difference between these two stages is the *mean range*.

See Index 814.5, Tides and Waves, for information on where tide and wave data may be obtained.

Figure 873.2A
Nomenclature of Tidal Ranges



Because of the great variation of tidal elements, Figure 873.2A was not drawn to scale.

The elevation of the design high tide may be taken as mean sea level (MSL) plus one-half the maximum tidal range (Rm).

(b) Design Wave Heights.

(1) General. Even for the simplest of cases, the estimation of water levels caused by meteorological conditions is complex. Elaborate numerical models requiring the use of a computer are available, but simplified techniques may be used to predict acceptable wind wave heights for the design of highway protection facilities along the shores of embayments, inland lakes, and reservoirs. It is recommended that for ocean shore protection designs the assistance of the U.S. Army Corp of Engineers be requested.

Shore protection structures are generally designed to withstand the wave that induces the highest forces on the structure over its economic service life. The design wave is analogous to the design storm considerations for determining return frequency. A starting point of reference for shore protection design is the maximum significant wave height that can occur once in about 20-years. Economic and risk considerations involved in selecting the design wave for a specific project are basically the same as those used in the analysis of other highway drainage structures.

(2) Wave Distribution Predictions. Wave prediction is called hindcasting when based on past meteorological conditions and forecasting when based on predicted conditions. The same procedures are used for hindcasting and forecasting. The only difference is the source of the meteorological data. Reference is made to the Army Corps of Engineers, Coastal Engineering Manual – Part II, for more complete information on the theory of wave generation and predicting techniques.

The prediction of wave heights from boat generated waves must be estimated from observations.

The surface of any large body of water will contain many waves differing in height, period, and direction of propagation. A representative wave height used in the design of bank and shore protection is the significant wave height, H_s. The significant wave height is the average height of the highest one-third of all the waves in a wave train for the time interval (return frequency) under consideration. Thus, the design wave height generally used is the significant wave height, H_s, for a 20-year return period.

Other design wave heights can also be designated, such as H_{10} and H_1. The H_{10} design wave is the average of the highest 10 percent of all waves, and the H_1 design wave is the average of the highest 1 percent of all waves. The relationship of H_{10} and H_1 to H_s can be approximated as follows:

$H_{10} = 1.27\ H_s$ and $H_1 = 1.67\ H_s$

Economics and risk of catastrophic failure are the primary considerations in designating the design wave average height.

(3) Wave Characteristics. Wave height estimates are based on wave characteristics that may be derived from an analysis of the following data:

- Wave gage records
- Visual observations
- Published wave hindcasts
- Wave forecasts
- Maximum breaking wave at the site

(4) Predicting Wind Generated Waves. The height of wind generated waves is a function of fetch length, windspeed, wind duration, and the depth of the water.

(a) Hindcasting -- The U.S. Army Corp of Engineers has historical records of onshore and offshore weather and wave observations for most of the California coastline. Design wave height predictions for coastal shore protection facilities should be made using this information and hindcasting methods. Deep-water ocean wave characteristics derived from offshore data analysis may need to be transformed to the project site by refraction and diffraction techniques. As mentioned previously, it is strongly advised that the Corps technical expertise be obtained so that the data are properly interpreted and used.

(b) Forecasting -- Simplified wind wave prediction techniques may be used to establish probable wave conditions for the design of highway protection on bays, lakes and other inland bodies of water. Wind data for use in determining design wind velocities and durations is usually available from weather stations, airports, and major dams and reservoirs.

The following assumptions pertain to these simplified methods:

- The fetch is short, 75 miles or less
- The wind is uniform and constant over the fetch.

It should be recognized that these conditions are rarely met and wind fields are not usually estimated accurately. The designer should therefore not assume that the results are more accurate than warranted by the accuracy of the input and simplicity of the method. Good, unbiased estimates of all wind generated wave parameters should be sought and the cumulative results conservatively interpreted. The individual input parameters should not each be estimated conservatively, since this may bias the result.

The applicability of a wave forecasting method depends on the available wind data, water depth, and overland topography. Water depth affects wave generation and for a given set of wind and fetch conditions, wave heights will be smaller and wave periods shorter if the wave generation takes place in transitional or shallow water rather than in deep water.

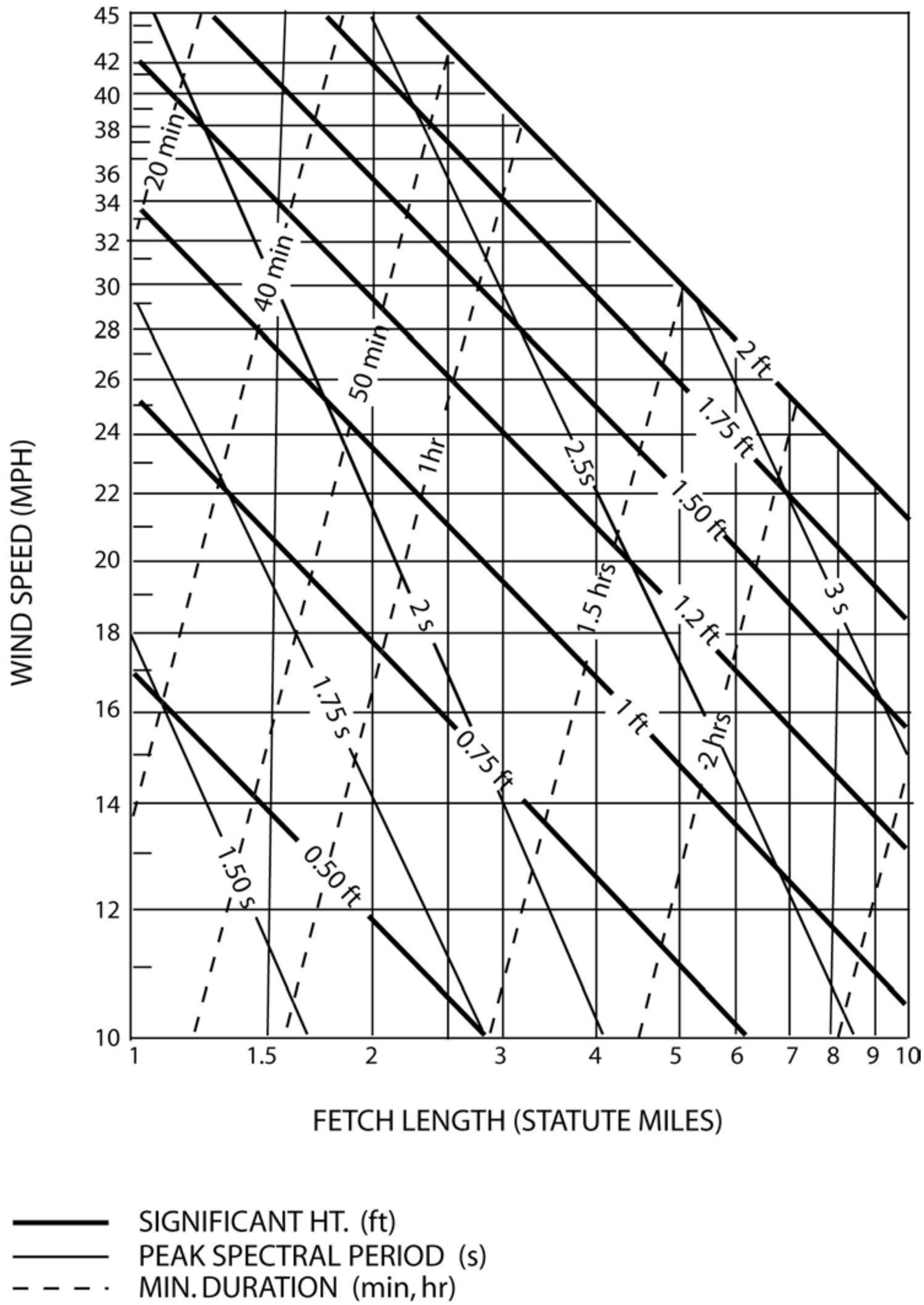
The height of wind generated waves may also be fetch-limited or duration-limited. Selection of an appropriate design wave may require a maximization procedure considering depth of water, wind direction, wind duration, wind-speed, and fetch length.

Procedures for predicting wind generated waves are complex and our understanding and ability to describe wave phenomena, especially in the region of the coastal zone, is limited. Many aspects of physics and fluid mechanics of wave energy have only minor influence on the design of shore protection for highway purposes. Designers interested in a more complete discussion on the rudiments of wave mechanics should consult the U.S.Army Corps of Engineers' Coastal Engineering Manual – Part II.

An initial estimate of wind generated significant wave heights can be made by using Figure 873.2B. If the estimated wave height from the nomogram is greater than 2 feet, the procedure may need to be refined. It is recommended that advice from the Army Corps of Engineers be obtained to refine significant wave heights, H_S, greater than 2 feet.

Figure 873.2B

Significant Wave Height Prediction Nomograph



(5) Breaking Waves. Wave heights derived from hindcasts or any forecasting method should be checked against the maximum breaking wave that the design stillwater level depth and nearshore bottom slope can support. The design wave height will be the smaller of either the maximum breaker height or the forecasted or hindcasted wave height.

The relationship of the maximum height of breaker which will expend its energy upon the protection, H_b, and the depth of water at the slope protection, d_s, which the wave must pass over are illustrated in Figure 873.2C.

The following diagram, with some specific references to the SPM, summarizes an overly simplified procedure that may be used for highway purposes to estimate wind generated waves and establish a design wave height for shore protection.

(6) Wave Run-up. Run-up is the extent, measured vertically, that an incoming wave will rise on a structure. An estimate of wave run-up, in addition to design wave height, will typically be needed and is required by policy for projects subject to California Coastal Commission (CCC) jurisdiction (see CCC guidance document "Beach Erosion and Response," December 1999). Procedures for estimating wave run-up for rough surfaces (e.g., RSP) are contained in the U.S. Army Corps of Engineers manual, Design of Coastal Revetments, Seawalls, and Bulkheads, (EM 1110-2-1614) published in 1995.

Procedures for estimating wave run-up for smooth surfaces (e.g., concrete paved slopes) and for vertical and curved face walls are contained in the U.S. Army Corps of Engineers, Shore Protection Manual, 1984. See Figure 873.2D for estimating wave run-up on smooth slopes for wave heights of 2 feet or less.

In protected bays and estuaries, waves generated by recreational or commercial boat traffic and other watercraft may dominate the design over wind generated waves. Direct observation and measurements during high tidal cycles may provide the designer the most useful tool for establishing wave run-up for these situations.

Determining Design Wave

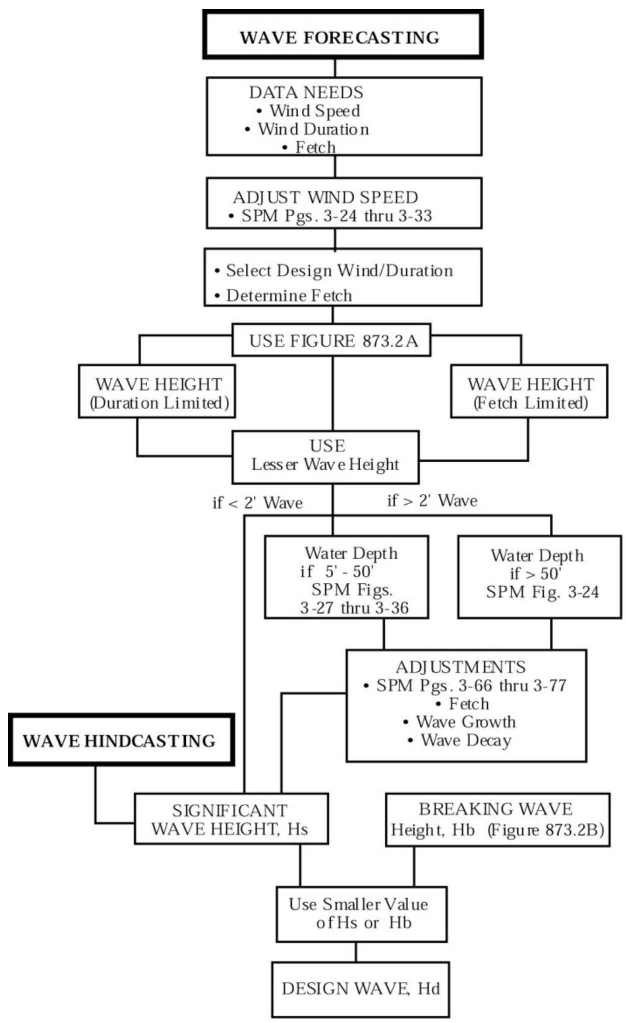
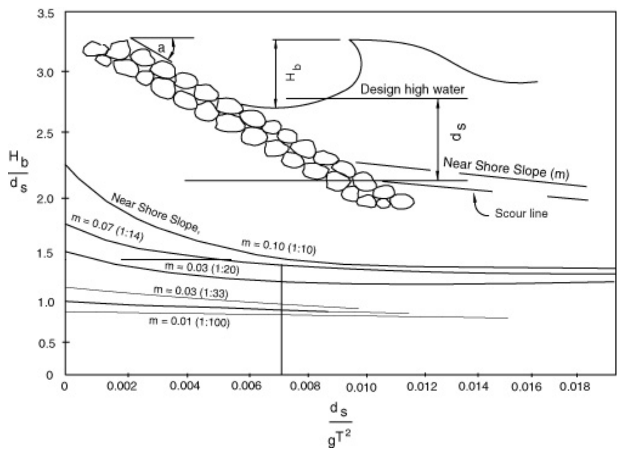


Figure 873.2C
Design Breaker Wave



<u>Example</u>
By using hindcast methods, the significant wave height (H_s) has been estimated at 4 feet with a 3 second period. Find the design wave height (H_d) for the slope protection if the depth of water (d) is only 2 feet and the nearshore slope (m) is 1:10.

<u>Solution</u>

$$\frac{d_s}{gT^2} = \frac{2 \text{ ft}}{(32.2 \text{ ft/s}^2) \times (3 \text{ sec})^2} = 0.007$$

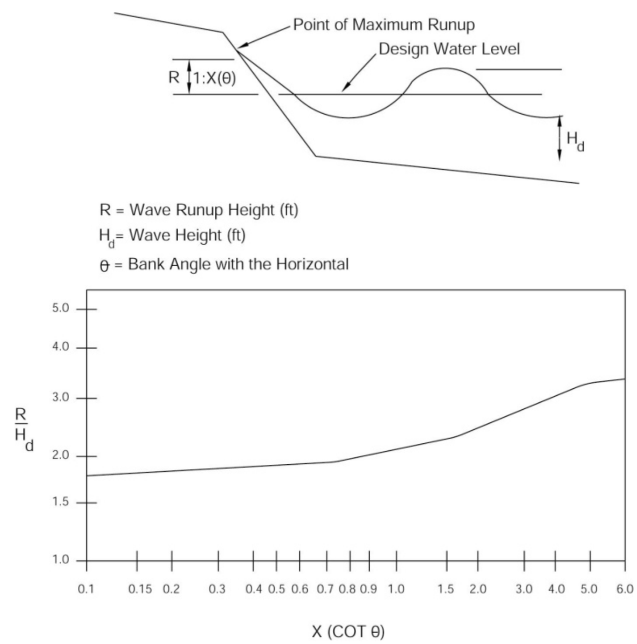
From Graph) - $H_b/d_s = 1.4$
$H_b = 2 \times 1.4 = 2.8$ ft

<u>Answer</u>
Since the maximum breaker wave height, H_b, is smaller than the significant deepwater wave height, H_s, the design wave height H_d is 2.8 feet.
T = Wave Period (SPM)

(c) Littoral Processes. Littoral processes result from the interaction of winds, waves, currents, tides, and the availability of sediment. The rates at which sediment is supplied to and removed from the shore may cause excessive accretion or erosion that can affect the structural integrity of shore protection structures or functional usefulness of a beach. The aim of good shore protection design is to maintain a stable shoreline where the volume of sediment supplied to the shore balances that which is removed.

Designers interested in a more complete discussion on littoral processes should consult the U.S. Army Corps of Engineers' Coastal Engineering Manual (CEM) – Part III.

Figure 873.2D
Wave Run-up on Smooth Impermeable Slope



873.3 Armor Protection

(1) *General.* Armor is the artificial surfacing of bed, banks, shore or embankment to resist erosion or scour. Armor devices can be flexible (self adjusting) or rigid.

Hard armoring of stream banks and shorelines, primarily with rock slope protection (RSP), has been the most common means of providing long-term protection for transportation facilities, and most importantly, the traveling public. With many years of use, dozens of formal studies and thousands of constructed sites, RSP is the armor type for which there exists the most quantifiable data on performance, constructability, maintainability and durability, and for which there exist several nationally recognized design methods.

Due to the above factors, RSP is the general standard against which other forms of armoring are compared. The results of internal research led to the publication of Report No. FHWA-CA-TL-95-10, "California Bank and Shore Rock Slope Protection Design". Within that report, the methodology for RSP design adopted as the Departmental standard, is the California Bank and Shore, (CABS), layered design. The full report is available at the following website:

http://www.dot.ca.gov/hq/oppd/hydrology/hydroidx.htm.

This design method, which is applied with slight variation to ocean and lake shores vs. stream banks, and is also followed for concreted RSP designs, is the only protection method as of this writing that has been formally adopted by the Caltrans Bank and Shore Protection Committee. Section 72 of the Standard Specifications provides all construction and material specifications for RSP designs. While standards (i.e., Standard Plans, Standard Specifications and/or SSP's) do exist for some other products discussed in this Chapter (most notably for gabions, but also for certain rolled or mat-style erosion control products), their primary application is for relatively flat slope or shallow ditch erosion control (gabions are also used as an earth retaining structure, see Topic 210 for more details).

Other armor types listed below and described throughout this Chapter are viable and may be used, upon approval of the Headquarters Hydraulic Engineer or Caltrans Bank and Shore Protection Committee, where conditions warrant. Although the additional step of headquarters approval of these non-standard designs is required, designers are encouraged to consider alternative designs, particularly those that incorporate vegetation or products naturally present in stream environments. The District Landscape Architect can provide design assistance together with specifications and details for the vegetative portion of this work.

(a) Flexible Types.

- Rock slope protection.
- Broken concrete slope protection.
- Broken concrete, uncoursed.
- Gabions, Standard Plan D100A and D100B.
- Precast concrete articulated blocks.
- Rock filled cellular mats.

(b) Rigid Types.

- Concreted-rock slope protection.
- Sacked concrete slope protection.
- Concrete slope protection.
- Concrete filled fabric slope protection.
- Air-blown mortar.
- Soil cement slope protection.

(c) Other Armor types:

(1) Channel Liners and Vegetation. Temporary channel lining can be used to promote vegetative growth in a drainage way or as protection prior to the placement of permanent armoring. This type of lining is used where an ordinary seeding and mulch application would not be expected to withstand the force of the channel flow. In addition to the following, other suitable products of natural or synthetic materials are available that may be used as temporary or permanent channel liners.

- Excelsior
- Jute
- Paper mats
- Fiberglass roving
- Geosynthetic mats or cells
- Pre-cast concrete blocks with open cells
- Brush layering
- Rock riprap in sizes smaller than backing No. 3

(2) Bulkheads. The bulkhead types are steep or vertical structures, like retaining walls, that support natural slopes or constructed embankments which include the following:

- Gravity or pile supported concrete or masonry walls.
- Crib walls

- Sheet piling
- Sea Walls

(d) General Design Criteria. In selecting the type of flexible or rigid armor protection to use the following characteristics are important design considerations.

(1) The lower limit, or toe, of armor should be below anticipated scour or on bedrock. If for any reason this is not economically feasible, a reasonable degree of security can be obtained by placement of additional quantities of heavy rock at the toe which can settle vertically as scour occurs.

(2) In the case of slope paving or any expensive revetment which might be seriously damaged by overtopping and subsequent erosion of underlying embankment, extension above design high water may be warranted. The usual limit of extension for streambank protection above design high water is 1 foot to 2 feet in unconstricted reaches and 2 feet to 3 feet in constricted reaches.

(3) The upstream terminal can be determined best by observation of existing conditions and/or by measuring velocities along the bank.

The terminal should be located to conform to outcroppings of erosion-resistant materials, trees, shrubs or other indications of stability.

In general, the upstream terminal on bends in the stream will be some distance upstream from the point of impingement or the beginning of curve where the effect of erosion is no longer damaging.

(4) When possible the downstream terminal should be made downstream from the end of the curve and against outcroppings, erosion-resistant materials, or returned securely into the bank so as to prevent erosion by eddy currents and velocity changes occurring in the transition length.

(5) The encroachment of embankment into the stream channel must be considered with respect to its effect on the conveyance of the stream and possible damaging effect on properties upstream due to backwater and downstream due to increased stream velocity or redirected stream flow.

(6) A smooth surface will generally accelerate velocity along the bank, requiring additional treatment (e.g., extended transition, cut-off wall, etc.) at the downstream terminal. Rougher surfaces tend to keep the thread of the stream toward the center of the channel.

(7) Heavy-duty armor used in exposures along the ocean shore may be influenced or dictated by economics, or the feasibility of handling heavy individual units.

(2) Flexible Revetments.

(a) Streambank Rock Slope Protection.

(1) General Features. This kind of protection, commonly called riprap, consists of rock courses placed upon the embankment or the natural slope along a stream. Rock, as a slope protection material, has a number of desirable features which have led to its widespread application.

It is usually the most economical type of revetment where stones of sufficient size and quality are available, it also has the following advantages:

- It is flexible and is not impaired nor weakened by slight movement of the embankment resulting from settlement or other minor adjustments.
- Local damage or loss is easily repaired by the addition of similar sized rock where required.
- Construction is not complicated and special equipment or construction practices are not usually necessary. (Note that Method A placement of

very large rock may require large cranes or equipment with special lifting capabilities).

- Appearance is natural, and usually acceptable in recreational and scenic areas.
- If exposed to fresh water, vegetation may be induced to grow through the rocks adding structural value to the embankment material and restoring natural roughness.
- Additional thickness (i.e., mounded toe design) can be provided at the toe to offset possible scour when it is not feasible to found it upon bedrock or below anticipated scour.
- Wave run-up is less than with smooth types (See Figure 873.2D).
- It is salvageable, may be stockpiled and reused if necessary.

In designing the rock slope protection for a given embankment the following determinations are to be made for the typical section.

- Depth at which the stones are founded (bottom of toe trench).
- Elevation at the top of protection.
- Thickness of protection.
- Need for geotextile and backing material.
- Face slope.

(a) Placement -- Two different methods of placement for rock slope protection are allowed under Section 72 of the Standard Specifications: Placement under Method A requires considerable care, judgment, and precision and is consequently more expensive than Method B. Method A should be specified primarily where large rock is required, but also for relatively steeper slopes.

Under some circumstances the costs of placing rock slope protection with refinement are not justified and Method B placement can be specified. To compensate for a partial loss and assure stability and a reasonably secure protection, the thickness is increased over the more precise Method A by 25 percent.

(b) Foundation Treatment -- The foundation excavation must afford a stable base on bedrock or extend below anticipated scour.

Terminals of revetments are often destroyed by eddy currents and other turbulence because of nonconformance with natural banks. Terminals should be secured by transitions to stable bank formations, or the end of the revetment should be reinforced by returns of thickened edges.

While a significant amount of research is currently being conducted, few methods exist for estimating scour along stream banks. One of the few is the method contained in the CHANLPRO Program developed by the U.S. Army Corps of Engineers. Based on the flume studies at the Corps' Waterways Experiment Station, the program is primarily used by the Corps for RSP designs on streams with 2 percent or lesser gradients, but contains an option for scour depth estimates in bends for sand channels. CHANLPRO is available at the following USACE website: http://chl.erdc.usace.army.mil/CHL.aspx?p=s&a=Software;3 along with a user guide containing equations, charts, assumptions and limitations to the method and example problems.

(c) Embankment Considerations -- Embankment material is not normally carried out over the rock slope protection so that the rock becomes part of the fill. With this type of construction fill material can filter down through the voids of the large stones and that portion of the fill above the rocks could be lost. If it is necessary to carry embankment material out over the rock slope protection a geotextile is required to prevent the loses of fill material.

The embankment fill slope is usually determined from other considerations such as the angle of repose for embankment material, or the normal 1V:4H specified for high-standard roads. If the necessary size of rock for the given exposure is not locally available, consideration should be given to flattening of the embankment slope to allow a smaller size stone, or substitution of other types of protection. On high embankments, alternate sections on several slopes should be compared, practically and economically; flatter slopes require smaller stones in thinner sections, but at the expense of longer slopes, a lower toe elevation, increased embankment, and perhaps additional right of way.

Where the roadway alignment is fixed, slope flattening will often increase embankment encroachment into the stream. When such an encroachment is environmentally or technically undesirable, the designer should consider various vertical, or near vertical, wall type alternatives to provide adequate stream width, allowing natural channel migration and the opportunity for enhancing habitat.

(d) Rock Slope Protection Fabric and Inner Layers of Rock -- The layered method of designing RSP installations was developed prior to widespread availability of the rock slope protection fabrics which are described in Standard Specification Section 88. The RSP fabric and multiple layers of rock ensure that fine soil particles do not migrate through the RSP due to hydrostatic forces and, thus, eliminate the potential for bank failure. The use of RSP fabric provides an inexpensive layer of protection retaining embankment fines in lieu of placing backing No. 3 or similar small, well graded materials. See Index 873.3(2)(a)(1)(e) "Gravel Filter."

Under special circumstances, the designer may consider allowing holes to be cut in the RSP fabric, generally to facilitate more rapid/extensive rooting of woody vegetation through the RSP revetment. This practice is only necessary for deeply rooted plant species. Holes in RSP fabric should not be cut below the stage of the 2-year return period event. The District Hydraulic Unit should be consulted for advice prior to any determination to cut or otherwise modify standard installation of RSP fabric.

Additionally, stronger and heavier RSP fabrics than those listed in the Standard Specifications are manufactured. They are used in special designs for larger than standard RSP sizes, or emergency installations where placement of the layered design is not feasible and large RSP must be placed directly on the fabric. These heavy weight fabrics have unit weights of up to 16 ounces per square yard. Contact the Headquarters Hydraulic Engineer for assistance regarding usage applications of heavy weight RSP fabrics.

(e) Gravel Filter -- Generally RSP fabric should always be used unless there is a permit requirement for establishment of vegetation that precludes the placement of fabric due to inadequate root penetration. Where RSP fabric cannot be placed, such as in stream environments where CA Fish & Game and NOAA Fisheries strongly discourage the use of RSP Fabric, a gravel filter is usually necessary with most native soil conditions to stop fines from bleeding through the typical RSP classes.

When a gravel filter is to be placed, the designer is advised to work with the District Materials Office to get a recommendation for the necessary gradation to work effectively with both the native backfill and the base layer of the RSP that is being placed. Among the methods available for designing the gravel filter are the Terzaghi method, developed exclusively for situations where the native backfill is sand, and the Cisten-Ziems method, which is often used for a broad variety of soil types. Where streambanks must be significantly rebuilt and reconfigured with imported material before RSP placement, the designer must ensure that the imported material will not bleed through the designed gravel filter.

(2) Streambank Protection Design. In the lower reaches of larger rivers wave action resulting from navigation or wind blowing over long reaches may be much more serious than velocity. A 2 foot wave, for example, is more damaging than direct impingement of a current flowing at 10 feet per second.

Well designed streambank rock slope protection should:

- Assure stability and compatibility of the protected bank as an integral part of the channel as a whole.
- Connect to natural bank, bridge abutments or adjoining improvements with transitions designed to ease differentials in alignment, grade, slope and roughness of banks.
- Eliminate or ease local embayments and capes so as to streamline the protected bank.
- Consider the effects of backwater above constrictions, superelevations on bends, as well as tolerance of occasional overtopping.
- Not be placed on a slope steeper than 1.5H:1V. Flatter slopes (see Figure 873.3A) use lighter stones in a thinner section and encourage overgrowth of vegetation, but may not be permissible in narrow channels.
- Use stone of adequate weight to resist erosion, derived from Figure 873.3A.
- Prevent loss of bank materials through interstitial spaces of the revetment. Rock slope protection fabric and multiple layers of backing should be used.
- Rest on a good foundation on bedrock or extend below the depth of probable scour. If questionable, use heavy bed stones and provide a wide base section with a reserve of material to slough into local scour holes (i.e., mounded toe).
- Reinforce critical zones on outer bends subject to impinging flow, using heavier stones, thicker section, and deeper toe.
- Be constructed in two or more layers of rock sizes, with progressively smaller rock toward native bank to prohibit loss of soil fines.
- Be constructed of rock of such shape as to form a stable protection

structure of the required section. Rounded boulders or cobbles must not be used on prepared ground surfaces having slopes steeper than 2.5H:1V

(a) Stone Size -- Where stream velocity governs, rock size may be estimated by using the nomogragh, Figure 873.3A.

The nomograph is derived from the following formula:

$$W = \frac{0.00002V^6 sg_r \csc^3(\beta - \alpha)}{(sg_r - 1)^3}$$

Where:

- sg_r = specific gravity of stones
- α = angle of face slope from the horizontal
- β = 70 for broken rock, a constant
- W = weight of minimum stable stone in lbs
- V = 2/3 average stream velocity, fps (flow parallel to bank) or 4/3 average stream velocity, fps (flow impinging on bank)

Where wave action is dominant, design of rock slope protection should proceed as described for shore protection.

(b) Design Height -- The top of rock slope protection along a stream bank should be carried to the elevation of the design high water plus some allowance for freeboard. The flood stage elevation adopted for design may be based on an empirically derived frequency of recurrence (probability of exceedance) or historic high water marks. This stage may be exceeded during infrequent floods, usually with little or no damage to the upper slope.

Design high water should not be based on an arbitrary storm frequency alone, but should consider the cost of carrying the protection to this height, the probable duration and damage if overtopped, and the importance of the facility.

When determining freeboard, or the height above design high water from which the RSP is to extend, one should consider: the size and nature of debris in the flow; the resulting potential for damage to the bank, the potential for streambed aggradation; and the confidence in data used to estimate design highwater. Freeboard may also be affected by regulatory or local agency requirements. Freeboard may be more generous along freeways, on bottleneck routes, on the outside bends of channels, or around critical bridges.

Design high water should be adjusted to the site based on sound engineering judgment.

Design Example -- The following example reflects the CABS method for designing RSP as described in Report No. FHWA – CA – TL – 95 – 10, as well as identify some of the considerations and technical principles that the designer must address to complete the installation design. These same considerations and principles apply to concreted RSP as well as RSP placed on beaches and shores (which are covered later), and therefore, separate examples for those designs are not provided. The designer is encouraged to review the entire report referenced above, available on the Division of Design website, for a comprehensive discussion of the basis of the CABS method and

Figure 873.3A

Nomograph of Stream-Bank Rock Slope Protection

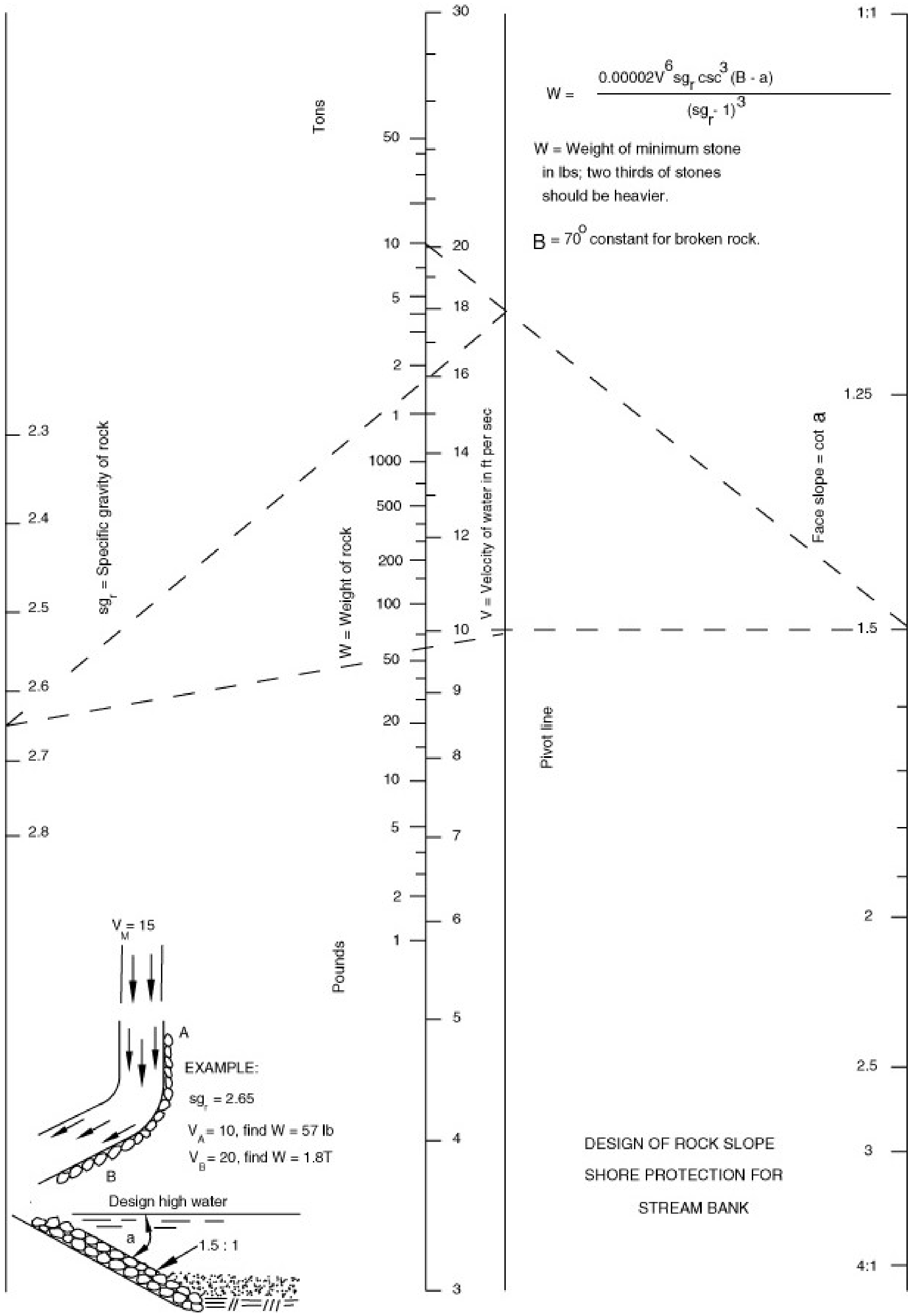
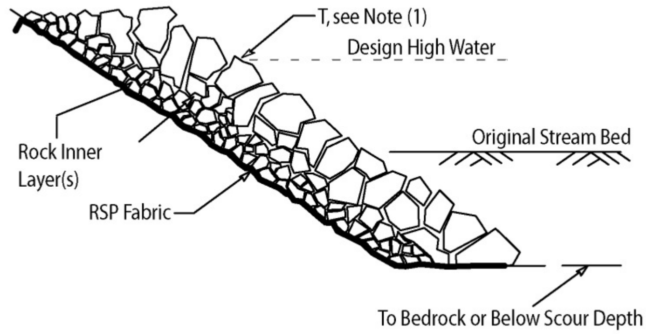
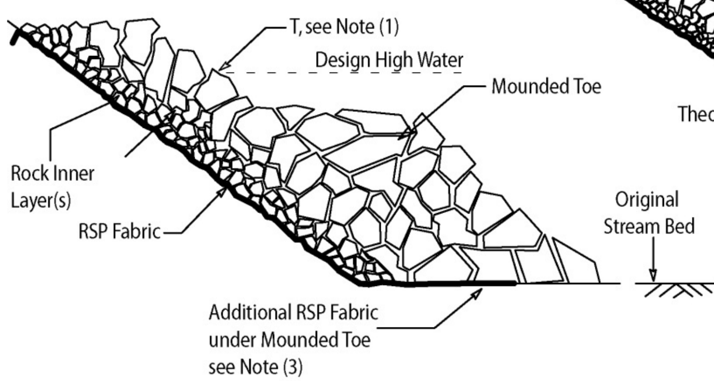


Figure 873.3C
Rock Slope Protection

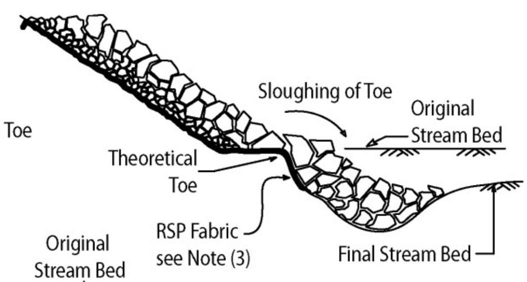
Embedded Toe RSP



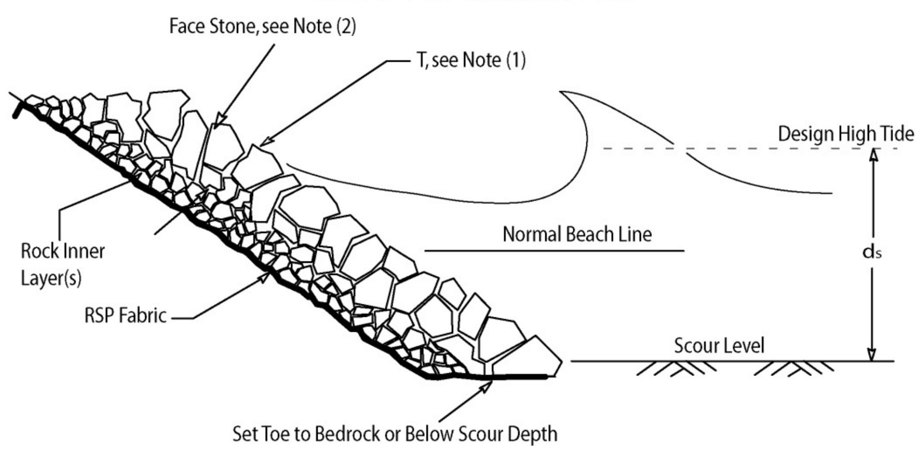
Mounded Toe RSP
(as constructed)



Mounded Toe RSP
(after launching of Mounded Toe)



Shore Protection RSP



NOTES:

(1) Thickness "T" from Table 873.3 C.
(2) Face stone is determined from Figure 873.3G.
(3) RSP fabric not to extend more than 20 percent of the base width of the Mounded Toe past the Theoretical Toe.

RSP design considerations. The following example assumes that the designer has conducted the appropriate site assessments and resulting calculations to establish average stream velocity, estimated depth of scour, stream alignment (i.e., parallel or impinging flow), length of stream bank to be protected and locations of natural hard points (e.g., rock outcroppings). Field reviews and discussions with maintenance staff familiar with the site are critical to the success of the design.

Given for example:

- Average stream velocity for design event – 16 feet per second
- Estimated scour depth – 5.5 feet
- Length of bank requiring protection – 550 feet
- Bank slope – 1.5:1
- Specific gravity of rock used for RSP – 2.65 (based on data from local quarry)
- Embankment is on outside of stream bend

1) Calculate minimum rock mass for outer layer:

$$W = \frac{(0.00002)\left(16 \times \frac{4}{3}\right)^{6}(2.65)}{(2.65-1)^{3}\sin^{3}(70-33.69)}$$

W = 5,350 lb

W = 2.67 ton = 2.43 tonne

NOTES:

For ease of computation with hand held calculators, cosecant has been converted to 1/sine.)

2) Select gradation for outer layer.

a) From minimum calculated rock weight of 2.67 tons in the example, select the rock weight from the left-side column tables in Standard Specification Section 72-2.02 that represents the standard rock weight just larger than the calculated weight. For ease, the Standard Specification tables are combined and reprinted in Table 873.3A.

The next larger rock mass above 2.67 ton is 4 ton. RSP this large is only to be installed using Method A placement techniques (i.e., individual rock placement, no end dumping). From this value, move horizontally across the gradation ranges to the “50-100” entry. From here, move vertically upward to select the design gradation, or RSP Class.

In this instance the name of the RSP class is 4 T.

b) Generally, this will represent the design outer RSP layer. However, the designer must assess this value against the site conditions observed during the field review and in conjunction with site history and projected future conditions prior to finalizing the selection. For the purposes of this example, we will assume this design gradation (i.e., 4 T RSP class) is appropriate.

Table 873.3A
Guide for Determining RSP-Class of Outside Layer

Standard Rock Sizes	GRADING OF ROCK SLOPE PROTECTION PERCENTAGE LARGER THAN											
	RSP-Classes [1]											
	Method A Placement					Method B Placement						
	RSP-Classes other than Backing									Backing No.		
	8 T	4 T	2 T	1 T	1/2 T	1 T	1/2 T	1/4 T	Light	1	2	3
16 ton	0-5											
8 ton	50-100	0-5										
4 ton	95-100	50-100	0-5									
2 ton		95-100	50-100	0-5		0-5						
1 ton			95-100	50-100	0-5	50-100	0-5					
1/2 ton				95-100	50-100	-----	50-100	0-5				
1/4 ton					95-100	95-100	-----	50-100	0-5			
200 lb							95-100	-----	50-100	0-5		
75 lb								95-100	-----	50-100	0-5	
25 lb									95-100	90-100	25-75	0-5
5 lb											90-100	25-75
1 lb												90-100

[1] “Facing” has same gradation as “Backing No. 1”. To conserve space “Facing” is not shown.

3) Determine RSP Layers. As previously discussed, properly designed RSP revetments are comprised of multiple layers of progressively smaller rock gradations progressing from the large sized rocks of the outer layer to the native soil or constructed embankment. Where the outer layer is composed of relatively small rock only a single inner layer may be needed. For a large rock outer layer as many as three inner layers may be required.

   For this example, the outer RSP layer is 4 T. From Table 873.3B, there are two options for the inner layers. The reason for multiple options for the larger RSP gradation classes is to allow the designer to better select RSP that is available from local quarry sources. Either set of layered designs is acceptable. The designer should contact rock producers in proximity to the project site to obtain price quotes for the different alternatives.

   This information may also be available from the District Materials Engineer. For the purposes of this example, we will select the layered design of: 4 T, 1 T, ¼ T, Backing No. 2 and Class 10 RSP Fabric.

4) Determine Thickness of Revetment. RSP layers are composed of rock classes shown in Table 873.3A. Each layer is at least 1.5 times the diameter of the median sized rock (D_{50}) in the gradation in order to prevent the smaller rocks in the lower layers from migrating.

   Table 873.3C provides the required thickness for the various RSP gradations and types of placement (Method A or Method B). Method B placement requires an increase in thickness to account for the looser rock contact and difficulty in controlling layer thickness inherent in end dumping of rock.

   Based on the table values, the total thickness of the design in our example (measured normal to the slope) is:

   4 T Layer = 6.8 ft

   1 T Layer = 4.3 ft

   ¼ T Layer = 3.3 ft

   Backing No. 2 Layer = 1.25 ft

   RSP Fabric = Effectively

   \+ 0.0 ft

   Total = 15.35 ft

5) Assess Stream Impact Due to Revetment. In some cases, the thickness of the completed RSP revetment creates a narrowing of the available stream channel width, to the extent that stream velocity or stage at the design event is increased to undesirable levels, or the opposite bank becomes susceptible to attack. In these cases, the bank upon which the RSP is to be placed must be excavated such that the constructed face of the revetment is flush with the original embankment.

6) Exterior Edges of Revetment. The completed design must be compatible with existing and future conditions. Freeboard and top edge of revetments were covered in

Table 873.3B

California Layered RSP

Outsider Layer RSP-Class *	Inner Layers RSP-Class *	Backing Class No. *	RSP-Fabric Class **
8 T	2 T over ½ T	1	10
8 T	1 T over ¼ T	1 or 2	10
4 T	½ T	1	10
4 T	1 T over ¼ T	1 or 2	10
2 T	½ T	1	10
2 T	¼ T	1 or 2	10
1 T	Light	None	8
1 T	¼ T	1 or 2	8
½ T	None	1	8
¼ T	None	1 or 2	8
Light	None	None	8
Backing No.1 ***	None	None	8

NOTES:

* Rock grading and quality requirements per Standard Specifications.

** RSP-fabric Type of geotextile and quality requirements per Section 88 Rock Slope Protection Fabric of the Standard Specifications. Class 8 RSP-fabric has lower weight per unit area and it also has lower toughness (tensile x elongation, both at break) than Class 10 RSP-fabric.

*** “Facing” RSP-Class has same gradation as Backing No. 1.

Table 873.3 C

Minimum Layer Thickness

RSP-Class Layer	Method of Placement	Minimum Thickness
8 T	A	8.5 ft
4 T	A	6.8 ft
2 T	A	5.4 ft
1 T	A	4.3 ft
½ T	A	3.4 ft
1 T	B	5.4 ft
½ T	B	4.3 ft
¼ T	B	3.3 ft
Light	B	2.5 ft
Facing	B	1.8 ft
Backing No. 1	B	1.8 ft
Backing No. 2	B	1.25 ft
Backing No. 3	B	0.75 ft

Index 873.3(2)(a)(2)(b) “Design Height.” For depth of toe, the estimated scour was given as 5.5 feet. This is the minimum toe depth to be considered. Again, based on site conditions and discussions with maintenance staff and others, determine if any long-term conditions need to be addressed. These could include streambed degradation due to local aggregate mining or headcutting. Regardless of the condition, the toe must be founded below the lowest anticipated elevation that could become exposed overthe service life of the embankment or roadway facility. As for the upstream and downstream ends, the given length of revetment is 500 feet. Again, this will

typically be a minimum, as the designer should seek natural rock outcroppings, areas of quiescent stream flow, or other inherently stable bank segments to end the RSP, see Figure 873.3D for example at ocean shore location.

Figure 873.3D
RSP Lined Ocean Shore

RSP placed at site subject to deep water wave attack. Terminal end of RSP tied into natural rock outcropping.

(b) Rock Slope Shore Protection.

(1) General Features. Rock slope protection when used for shore protection, in addition to the general advantages listed previously for streambank rock slope protection, reduces wave runup as compared to smooth types of protection.

(a) Method A placement is normally specified for ocean shore protection since very large stone is typically needed. Rock mass for lake shores and protected bays are often based on the height of boat generated waves.

(b) Foundation treatment in shore protection may be controlled by tidal action as well as excavation difficulties and production may be limited to only two or three toe or foundation rocks per tide cycle. If toe rocks are not properly bedded, the subsequent vertical adjustment may be detrimental to the protection above. Even though rock is self-adjusting, the bearing of one rock to another may be lost. It is often necessary to construct the toe or foundation to an elevation approximating high tide in advance of embankment construction to prevent erosion of the embankment.

(2) Shore Protection Design.

(a) Stone Size -- For waves that are shoaling as they approach the protection the required stone size may be determined by Using Chart B, Figure 873.3G.

The nomograph is derived from the following formula:

$$W = \frac{0.003 d_B{}^3 sg_r \csc^3(\beta - \alpha)}{\left(\frac{sg_r}{sg_w} - 1\right)^3}$$

Where:

- d_B = maximum depth in feet of water at toe of the rock slope protection, see Figure 873.3C
- sg_r = specific gravity of stones
- sg_w = specific gravity of water (sea water = 1.0265)
- α = angle of face slope from the horizontal
- β = 70 for broken rock, a constant
- W = weight of minimum stable stone in lbs

In general, d_B will be the difference between the elevation of the scour line at the toe and the maximum stillwater level. For ocean shore, d_s may be taken as the distance from the scour line to mean sea level plus one-half the maximum tidal range.

If the deep-water waves, see Figure 873.3D, reach the protection, the stone size may be determined by using Chart A, Figure 873.3G. The nomograph is derived from the following formula:

$$W = \frac{0.00231 H_d{}^3 sg_r \csc^3(\beta - \alpha)}{\left(\frac{sg_r}{sg_w} - 1\right)^3}$$

Where:

H_d = design wave in feet, see Index 873.2

If in doubt whether waves generated by fetch and wind velocity will be of sufficient size to be affected by shoaling, use both charts and adopt the smaller value.

(b) Dimensions -- Rock should be founded in a toe trench dug to hard rock or keyed into soft rock. If bedrock is not within reach, the toe should be carried below the estimated depth of probable scour. If the scour depth is questionable, additional thickness of rock may be placed at the toe which will adjust and provide deeper support. In determining the elevation of the scoured beach line the designer should observe conditions during the winter season, consult records, or ask persons who have a knowledge of past conditions.

Wave run-up is reduced by the rough surface of rock slope protection. In order that the wash will not top the rock, it should be carried up to an elevation of twice the maximum depth of water ($2d_s$) or to an elevation equal to the maximum depth of water plus the deep-water wave height ($d_s + H_d$), whichever is the *lower*. See Figure 873.3C.

Consideration should also be given to protecting the bank above the rock slope protection from splash and spray.

Design thickness of the protection should be based on the same procedures as used for streambanks. For typical conditions the thickness required for the various sizes are shown on Table 873.3B. Except for toes on questionable foundation, as explained above, additional thickness will not compensate for undersized stones. When properly constructed, the largest stones will be on the outside, and if the wave forces displace these, additional thickness will only add slightly to the time of failure. Shore revetments, particularly ocean shore locations, are often candidates for using a mounded toe design. Where it is not practical to excavate to bedrock or to the anticipated scour depth to set the revetment toe, an alternative treatment is to place additional rock (i.e., mound) of the same mass as the outer layer at the toe. The volume to be placed should be slightly greater than the amount that would have been needed to extend the toe to the estimated scour depth. See figure 873.3C for a depiction of a mounded toe installation.

As scour occurs at the toe of the revetment, this mounded rock will drop into the scour hole. It is important in mounded toe designs to require that excess RSP fabric be placed so that as the scour hole develops and rock begins to drop, the excess RSP fabric will "unfold" and also drop into place to limit loss of embankment.

(c) Gabions. Gabion revetments consist of rectangular wire mesh baskets filled with stone. See Standard Plan D100A and D100B for gabion basket details and the Standard Specifications for requirements.

Gabions are formed by filling commercially fabricated and preassembled wire baskets with rock. There are two types of gabions, wall type and mattress type. In wall type the empty cells are positioned and filled in place to form walls in a stepped fashion. Mattress type baskets are positioned on the slope and filled. Wall type revetment is not fully self adjusting but has some flexibility. The mattress type is very flexible. For some locations, gabions may be more aesthetically acceptable than rock riprap. Where larger stone sizes are not readily available and the flow does not abrade the wire baskets, they may also be more cost effective. However, caution is advised regarding in-stream placement of gabions, and some form of abrasion protection in the form of wooden planks or other facing will typically be necessary, see Figure 873.3E.

Figure 873.3E

Gabion Lined Streambank

Gabion wall with timber facing to protect wires from abrasive flow.

(d) Articulated Precast Concrete. This type of revetment consists of pre-cast concrete blocks which interlock with each other, are attached to each other, or butted together to form a continuous blanket or mat. A number of block designs are commercially available. They differ in shape and method of articulation, but share common features of flexibility and rapid installation. Most provide for establishment of vegetation within the revetment.

The permeable nature of these revetments permits free draining of the embankment and their flexibility allows the mat to adjust to minor changes in bank geometry. Pre-cast concrete block revetments may be economically justified where suitable rock for slope protection is not readily available. They are generally more aesthetically pleasing than other types of revetment, particularly after vegetation has become established.

Individual blocks are commonly joined together with steel cable or synthetic rope, to form articulated block mattresses. Pre-assembled in sections to fit the site, the mattresses can be used on slopes up to 2:1. They are anchored at the top of the revetment to secure the system against slippage.

Pre-cast block revetments that are formed by butting individual blocks end to end, with no physical connection, should not be used on slopes steeper than 3:1. An engineering fabric is normally used on the slope to prevent the erosion of the underlying embankment through the voids in the concrete blocks.

Refer to HEC-11, Design of Riprap Revetment, Section 6.2, and HEC-23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 4, for further discussion on the use of articulated concrete blocks.

(3) Rigid Revetments.

(a) Concreted-Rock Slope Protection.

(1) General Features. This type of revetment consists of rock slope protection with interior voids filled with PCC to form a monolithic armor. A typical section of this type of installation is shown in Figure 873.3F.

It has application in areas where rock of sufficient size for ordinary rock slope protection is not economically available.

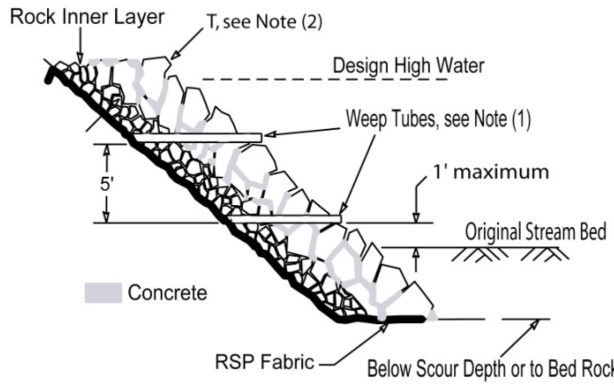
(2) Design Concepts. Concreting of RSP is a common practice where availability of large stones is limited, or where there is a need to reduce the total thickness of a RSP revetment. Inclusion of the concrete, and the labor required to place it, makes concreted RSP installations more expensive per unit area than non-concreted installations.

Design procedures for concreted RSP revetments are similar to that of non-concreted RSP. Start by following the design example provided in Index 873.3(2)(a)(2)(c) to select a stable rock size for a non-concreted design based on the site conditions. This non-concreted rock size is divided by a factor of roughly four or five to arrive at the appropriate size outer layer rock for a concreted revetment. The factor is based on observations of previously constructed facilities and represents the typical sized pieces that stay together even after severe cracking (i.e., failed revetments will still usually have segments of four to five rocks holding together). As with the non-concreted design procedures, use the rock size derived from this calculation to enter Table 873.3A (i.e., round up to the next larger rock mass, which will represent the 50-100 percentage larger than gradation range) and then select the appropriate RSP Class. The thickness and rock sizing of the inner layers can be based on the reduced sizing of the outer layer rock. Note that as shown in Figure 873.3F, the inner layers of rock are not concreted.

As this type of protection is rigid without high strength, support by the embankment must be maintained. Slopes steeper than the angle of repose of the embankment are risky, but with rocks grouted in place, little is to be gained with slopes flatter than 1.5:1. Precautions to prevent undermining of embankment are particularly important, see Figure 873.3H. The concreted-rock must be founded on solid rock or below the depth of possible scour. Ends should be protected by tying into stable rock or

Figure 873.3F

Concreted-Rock Slope Protection



NOTES:

(1) If needed to relieve hydrostatic pressure.

(2) Refer to Table 873.3 C for section thickness.

Dimensions and details should be modified as required.

forming smooth transitions with embankment subjected to lower velocities. As a precaution, cutoff stubs may be provided. If the embankment material is exposed at the top, freeboard is warranted to prevent overtopping.

Figure 873.3H
Toe Failure - Concreted RSP

Toe of concreted RSP that has been undermined.

The design intent is to place an adequate volume of concrete to tie the rock mass together, but leave the outer face roughened with enough rock projecting above the concrete to slow flow velocities to more closely approximate natural conditions.

The volume of concrete required is based on filling roughly two-thirds of the void space of the outer rock layer, as shown in Figure 873.3F. The concrete is rodded or vibrated into place leaving the outer stones partially exposed. Void space for the various RSP gradations ranges from approximately 30 percent to 35 percent for Method A placed rock to 40 percent to 45 percent for Method B placed rock of the total volume placed.

(2) Specifications. Quality specifications for rock used in concreted-rock slope protection are usually the same as for rock used in ordinary rock slope protection. However, as the rocks are protected by the concrete which surrounds them, specifications for specific gravity and hardness may be lowered if necessary. The concrete used to fill the voids is normally 1 inch maximum size aggregate minor concrete. Except for freeze-thaw testing of aggregates, which may be waived in the contract special provisions, the concrete should conform to the provisions of Standard Specification Section 90.

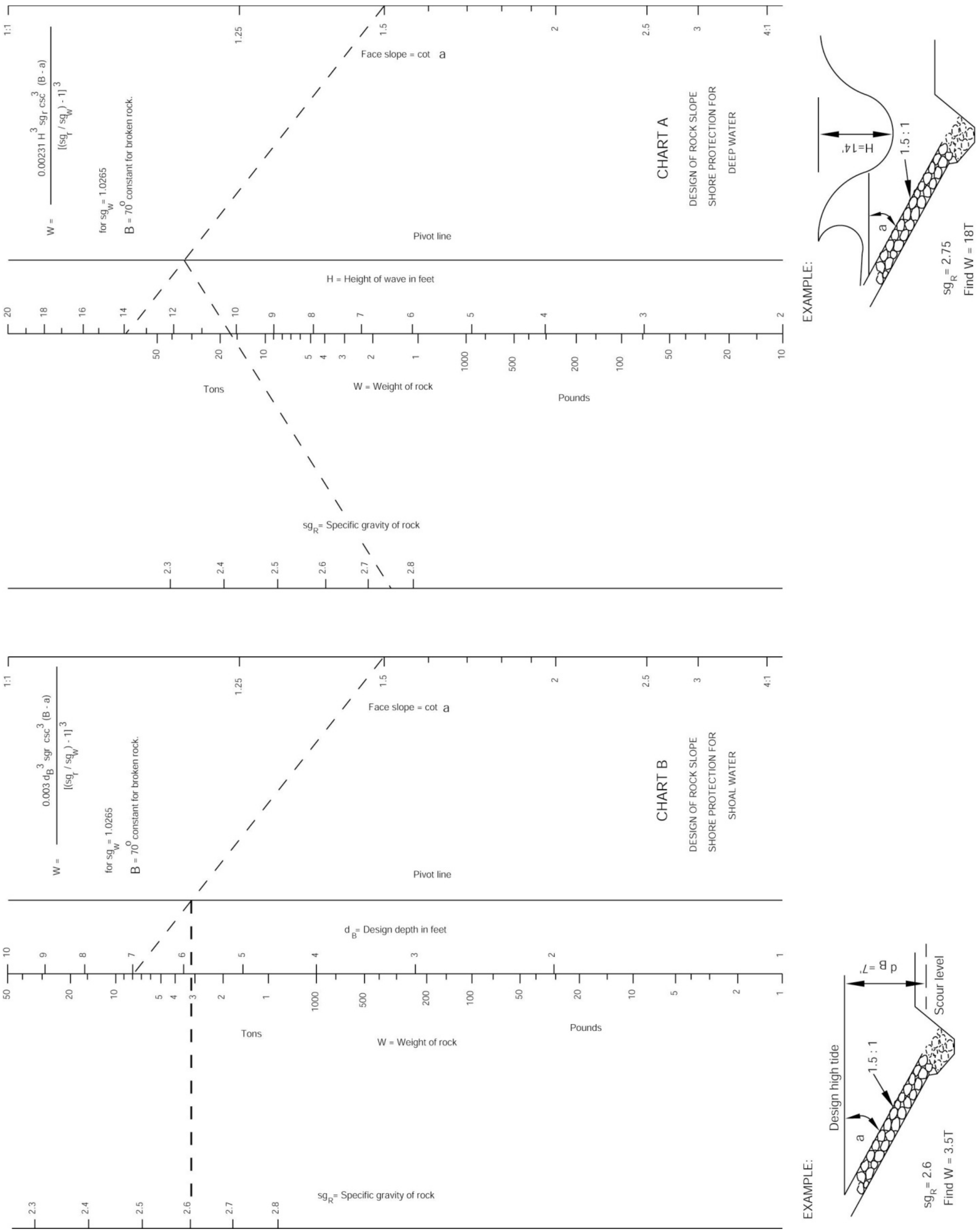
Size and grading of stone and concrete penetration depth are provided in Standard Specification Section 72.

(b) Sacked-Concrete Slope Protection. This method of protection consists of facing the embankment with sacks filled with concrete. It is expensive, but historically was a much used type of revetment. Much hand labor is required but it is simple to construct and adaptable to almost any embankment contour. Use of this method of slope protection is generally limited to replacement or repair of existing sacked concrete facilities, or for small, unique situations that lend themselves to hand-placed materials.

Tensile strength is low and as there is no flexibility, the installation must depend almost entirely upon the stability of the embankment for support and therefore should not be placed on face slopes much steeper than the angle of repose of the embankment material. Slopes steeper than 1:1 are rare; 1.5:1 is common. The flatter the slope, the less is the area of bond between sacks. From a construction standpoint it is not practical to increase the area of bond between sacks; therefore for slopes as flat as 2:1 all sacks should be laid as headers rather than stretchers.

Integrity of the revetment can be increased by embedding dowels in adjoining sacks to reinforce intersack bond. A No. 3 deformed bar driven through a top sack into the underlying sack while the concrete is still fresh is effective. At cold joints, the first course of sacks should be impaled on projecting bars that were driven into the last previously placed course. The extra

Figure 873.3G
Nomographs For Design of Rock Slope Shore Protection



strength may only be needed at the perimeter of the revetment.

Most failures of sacked concrete are a result of stream water eroding the embankment material from the bottom, the ends, or the top.

The bottom should be founded on bedrock or below the depth of possible scour.

If the ends are not tied into rock or other nonerosive material, cutoff returns are to be provided and if the protection is long, cutoff stubs are built at 30-foot intervals, in order to prevent or retard a progressive failure.

Protection should be high enough to preclude overtopping. If the roadway grade is subject to flooding and the shoulder material does not contain sufficient rock to prevent erosion from the top, then pavement should be carried over the top of the slope protection in order to prevent water entering from this direction.

Class 8 RSP fabric as described in Standard Specification Section 88 should be placed behind all sacked concrete revetments. For revetments over 4 feet in height, weep tubes should also be placed, see Figure 873.3F.

For good appearance, it is essential that the sacks be placed in horizontal courses. If the foundation is irregular, corrective work such as placement of entrenched concrete or sacked concrete is necessary to level up the foundation. Refer to HDS No. 6, Section 6.6.5, for further discussion on the use of sacked concrete slope protection.

(c) Concrete Slope Paving.

(1) General Features. This method of protection consists of paving the embankment with portland cement concrete. Slope paving is used only where flow is controlled and will not over-top the protection.

It is particularly adaptable to locations where high-velocity flow is not detrimental but desirable and the hydraulic efficiency of smooth surfaces is important. It has been used very little in shore protection. On a cubic feet basis the cost is high but as the thickness is generally only 3 inches to 6 inches, the cost on a basis of area covered will usually be less than for sacked-concrete slope protection. This is especially so when sufficiently large quantities are involved and alignment is such as to warrant the use of mass production equipment such as slip-form pavers.

Due to the rigidity of PCC slope paving, its foundation must be good and the embankment stable. Although reinforcement will enable it to bridge small settlements of the embankment face, even moderate movements could lead to cracking of the paving or failure. The toe must be on bedrock or extend below possible scour. When this is not feasible without costly underwater construction, rock or PCC grouted RSP have been used as a foundation. A better but much more expensive solution is to place the toe on a PCC wall or piles.

Every precaution must be taken to exclude stream water from pervious zones behind the slope paving. The light slabs will be lifted by comparatively small hydrostatic pressures, opening joints or cracks at other points in a series of progressive failures leading to extensive or complete failure.

Considering the severity of failure from bank erosion or hydrostatic pressure after overtopping, 1 foot to 2 feet of freeboard above design high water is recommended for this type of revetment. Refer to HEC-11, Design of Riprap Revetment, Section 6.4, for further discussion on the use of concrete slope paving. Table 873.3D gives channel lining thickness.

(4) Bulkheads. A bulkhead is a steep or vertical structure supporting a natural slope or

constructed embankment. As bank and shore protection structures, bulkheads serve to secure the bank against erosion as well as retaining it against sliding. As a slope protection structure, revetment design principles are used, the only essential difference being the steepness of the face slope. As a retaining structure, conventional design methods for retaining walls, cribs and laterally loaded piles are used.

Bulkheads are usually expensive, but may be economically justified in special cases where valuable riparian property or improvements are involved and foundation conditions are not satisfactory for less expensive types of slope protection. They may be used for toe protection in combination with other revetment types of slope protection. Some other considerations that may justify the use of bulkheads include:

- Encroachment on a channel cannot be tolerated.
- Retreat of highway alignment is not viable.
- Right of Way is restricted.
- The force and direction of the stream can best be redirected by a vertical structure.

The foundation for bulkheads must be positive and all terminals secure against erosive forces. The length of the structure should be the minimum necessary, with transitions to other less expensive types of slope protection when possible. Eddy currents can be extremely damaging at the terminals and transitions. If overtopping of the bulkheads is anticipated, suitable protection should be provided.

Along a stream bank, using a bulkhead presumes a channel section so constricted as to prohibit use of a cheaper device on a natural slope. Velocity will be unnaturally high along the face of the bulkhead, which must have a fairly smooth surface to avoid compounding the restriction. The high velocity will increase the threat of scour at the toe and erosion at the downstream end. Allowance must be made for these threats in selecting the type of foundation, grade of footing, penetration of piling, transition, and anchorage at downstream end. Transitions at both ends may appropriately taper the width of channel and slope of the bank. Transition in roughness is desirable if attainable. Refer to HDS No. 6, Section 6.4.8, for further discussion on the use of bulkheads to prevent streambank erosion or failure.

Along a shore, use of a bulkhead presumes a steep lake or sea bed profile, such that revetment on a 1.5:1 or flatter slope would project into prohibitively deep water or permit intolerable wave runup. Such shores are generally rocky, offering good foundation on residual reefs, but historic destruction of the overlying formation attests to the hydraulic power of the sea to be resisted by an artificial replacement. The face of such a bulkhead must be designed to absorb or dissipate as much as practical the shock of these forces. Designers should consult the U.S. Army Corps of Engineers EM-1110-2-1614, Design of Coastal Revetments, Seawalls, and Bulkheads, for more complete information and details.

(a) Concrete or Masonry Walls. The expertise and coordination of several engineering disciplines is required to accomplish the development of PS&E for concrete walls serving the dual purpose of slope protection and support. The Division of Structures is responsible for the structural integrity of all retaining walls, including bulkheads.

(b) Crib walls. Timber and concrete cribs can be used for bulkheads in locations where some flexibility is desirable or permissible. Metal cribs are limited to support of embankment and are not recommended for use as protection because of vulnerability to corrosion and abrasion.

The design of crib walls is essentially a determination of line, foundation grade, and height with special attention given to potential scour and possible loss of backfill at the base and along the toe. Design details for concrete crib walls are shown on Standard Plans C7A through C7G. Concrete crib walls used as bulkheads and exposed to salt water require special provisions specifying the use of coated rebars and special high density concrete.

Recommendations from METS Corrosion Technology Branch should be requested.

Design details for timber crib walls of dimensioned lumber are shown on Standard Plans C9A and C9B. Timber cribs of logs, notched to interlock at the contacts, may also be used. All dimensioned lumber should be treated to resist decay.

(c) Sheet Piling. Timber, concrete and steel sheet piling are used for bulkheads that depend on deep penetration of foundation materials for all or part of their stability. High bulkheads are usually counterforted at upper levels with batter piles or tie back systems to deadmen. Any of the three materials is adaptable to sheet piling or a sheathed system of post or column piles.

Excluding structural requirements, design of pile bulkheads is essentially as follows:

- Recognition of foundation conditions suitable to or demanding deep penetration. Penetration of at least 15 feet below scour level, or into soft rock, should be assured.
- Choice of material. Timber is suitable for very dry or very wet climates, for other situations economic comparison of preliminary designs and alternative materials should be made.
- Determination of line and grade. Fairly smooth transitions with protection to high-water level should be provided.

(5) *Vegetation*. Vegetation is the most natural method for stabilization of embankments and channel bank protection. Vegetation can be relatively easy to maintain, visually attractive and environmentally desirable. The root system forms a binding network that helps hold the soil. Grass and woody plants above ground provide resistance to the near bank water flow causing it to lose some of its erosive energy.

Erosion control and revegetation mats are flexible three-dimensional mats or nets of natural or synthetic material that protect soil and seeds against water erosion prior to establishment of vegetation. They permit vegetation growth through the web of the mat material and have been used as temporary channel linings where ordinary seeding and mulching techniques will not withstand erosive flow velocities. The designer should recognize that flow velocity estimates and a particular soils resistance to erosion are parameters that must be based on specific site conditions. Using arbitrarily selected values for design of vegetative slope protection without consultation with the District Hydraulic Unit and/or the District Landscape Architect Unit is not recommended. However, a suggested starting point of reference is Table 865.2 in which the resistance of various unprotected soil classifications to flow velocities are given. Under near ideal conditions, ordinary seeding and mulching methods cannot reasonably be expected to withstand sustained flow velocities above 4 feet per second. If velocities are in excess of 4 feet per second, a lining maybe needed, see Table 873.3E.

Temporary channel liners are used to establish vegetative growth in a drainage way or as slope protection prior to the placement of a permanent armoring. Some typical temporary channel liners are:

- Straw
- Excelsior
- Jute
- Woven paper

Vegetative and temporary channel liners are suitable for conditions of uniform flow and moderate shear stresses.

Permanent soil reinforcing mats and rock riprap may serve the dual purpose of temporary and permanent channel liner. Some typical permanent channel liners are:

- Gravel or cobble size riprap
- Fiberglass roving
- Geosynthetic mats
- Polyethelene cells or grids
- Gabion Mattresses

Table 873.3E

Permissible Velocities for Flexible Channel Linings

Type of Lining [1]	Permissible Velocity (ft/s)	
	Intermittent Flow	Sustained Flow
Vegetation:		
Bermuda Grass, uncut	4.0	2.5
Bermuda Grass, mowed or Crab Grass, uncut	4.0	2.5
Riprap:		
Gravel, 1 in	3.0	2.0
Gravel, 2 in	3.5	2.5
Cobble, 3 in	5.0	4.0
Cobble, 6 in	7.5	6.5
Temporary:		
Woven Paper Net	4.5	3.5
Jute Net	5.0	4.0
Fiberglass Roving	5.5	4.5
Straw with Net	6.5	4.5
Curled Wood Mat	6.5	4.5
Synthetic Mat	10.5	7.5

NOTE:

(1) Ref. HEC-15.

However, geosynthetics and plastic (polyethylene, polypropylene, polyamide, etc.) based mats must be installed in a fashion where there will be no potential for long-term sunlight exposure, as these products will degrade due to UV radiation.

Composite designs are often used where there are sustained low flows of high to moderate velocities and intermediate high water flows of low to moderate velocities. Brush layering is a permanent type of erosion control technique that may also have application for channel protection, particularly as a composite design.

Additional design information on vegetation, and temporary and permanent channel liners is given in Chapter IV, HEC-15, Design of Roadside Channels and Flexible Linings.

873.4 Training Systems

(1) *General*. Training systems are structures, usually within a channel, that act as countermeasures to control the direction, velocity, or depth of flowing water. As shore protection, they control shoaling and scour by deflecting the strength of currents and waves.

The degree of permeability is among the most important properties of control structures. An impermeable structure may deflect a current entirely, whereas a permeable structure may serve mainly to reduce the strength of water velocity, currents or waves.

Training systems of the retard and permeable jetty types are similar in that they are usually extensive or multi-unit open structures like; piling, fencing, and unit frames. They are dissimilar in function and alignment, retards being parallel and groins oblique to the banks. The retard is a milder remedy than jetty construction.

(a) Retard Types. A retard is a bank protection structure designed to check riparian velocity and induce silting and accretion. They are usually placed parallel to the highway embankment or erodible banks of channels on stable gradients. Retards typically take the following forms of construction:

- Fencing - single or double lines
- Palisades - piles and netting
- Timber piling or pile bents
- Steel or timber jacks

Retards are applicable primarily on streams which meander to some extent within a mature valley. Typical uses include the following:

- Protection at the toe of highway embankments that encroach on a stream channel.
- Training and control to inhibit erosion upstream and downstream from stream crossings.
- Control of erosion redeposition of material where progressive embayments are creating a problem.

(1) Fence Type. Fence-type structures are used as retards, permeable or impermeable jetties, and as baffles. These structures can be constructed of various materials.

Fence type retards may be effective on smaller streams and areas subject to infrequent attack, such as overflow areas. Single and double rows of various types of fencing have been used. The principal difference between fence retards and ordinary wire fences is that the posts of retards must be driven sufficiently deep to avoid loss by scour.

Permeability can be varied in the design to fit the requirements of the location for single fences, the factor most readily varied is the pattern of the wire mesh. For multiple fences, the mesh pattern can be varied or the space between fences can be filled to any desired height. Making optimum use of local materials, this fill may be brush ballasted by rock, or rock alone.

(2) Piles and Palisades. Retards and jetties may be of single, double, or triple rows of piles with the outside or upstream row faced with wire mesh fencing material, boards or polymeric straps

interwoven into a high-strength net. The facing adds to the retarding effect and may trap light brush or debris to supplement its purpose. This type retard is particularly adapted to larger streams where the piles will remain in the water. The number of pile rows and amount of facing may be varied to control the deposition of material. In leveed rivers it is often desirable to discourage accretion so as to not constrict the channel but provide sufficient retarding effect to prevent loss of a light bank protection such as vegetation or light rock facing.

Typical design considerations include:

- If the stream carries heavy debris, the elevation of the top of the pile should be well below the high-water level in order that heavy objects such as logs will pass over the top during normal floods.
- Piles must have sufficient penetration to prevent loss from scour or impact by floating debris or both. This is especially important for the piles at the outer end of jetties. If scour is a problem, the pile may be protected by a layer of rock placed on the streambed. Piles should be long enough to penetrate below probable scour, with penetration of a least 15 feet in streams with sandy beds and velocities of 10 feet per second to 15 feet per second.
- Ends of the system should be joined to the bank in order to prevent parallel high-velocity flow between the retard and the bank. If the installation is long, additional bank connections may be placed at intervals.
- Facing material should be fastened to the upstream or channel side of the piling in order that the force of the water and impact of debris will not be entirely on the fasteners.

(3) Jacks and Tetrahedrons. Jacks and tetrahedrons are skeletal frames that can be used as retards or permeable jetties. Cables can be used to tie a number of similar units together in longitudinal alignment and for anchorage of key units to deadmen. Struts and wires are added to the basic frames to increase impedance to flow of water directly by their own resistance and indirectly by the debris they collect.

Both devices serve best in meandering streams which carry considerable bed load during flood stages. Impedance of the stream along the string of units will cause deposit of alluvium, especially at the crest and during the falling stage. Beds of such streams often scour on the rising stage, undercutting the units and causing their subsidence, often accompanied by rotation when one leg or side is undercut more than the other. Deposition of the falling stage usually restores the former bed, partially or completely burying the units. In that lowered and rotated position, they may still be completely effective in future floods.

Retards may be used alone or in combination with other types of slope protection. In combination with a lighter type of armor they may be more economical than a heavier type of protection. They can be used as toe protection for other types of slope protection where a good foundation is impractical because of high water or extreme depth of poor material.

Where new embankment is placed behind the retard consideration should be given to protecting the slope to inhibit erosion until the retard has had an opportunity to function. The slope protection used should promote the establishment of a natural cover, such as discussed under Index 873.3(5), Vegetation.

Retards on tangent reaches of narrow channels may, by slowing the velocity on one side, cause an increase in velocity, on the other. On wider reaches of a meandering stream they may, by slowing a rebounding high velocity thread, have a beneficial effect on the opposite bank. Where the prime purpose of the retard system is to reduce stream bank velocity to encourage deposition of material intended to alter the channel alignment the effect on adjacent property must be assessed. Where deposition of material is the primary function, the service life of the installation is dependent on the deposition rate and the ultimate establishment of a natural retard.

The length of a retard system should extend from a secure anchorage on the upstream end to anchorage on the downstream end beyond the area under direct attack. Since erosion often progresses downstream, this possibility should be considered in determining the planned length.

The top of a retard need not extend to the elevation of design high water. In major rivers and streams where drift is large and heavy it is essential that the retard be low enough to pass debris over the top during stages of high flow.

For further information on retards, refer to Section 6.4.4 of HDS No. 6.

(b) Jetty Types. A jetty is an elongated artificial obstruction projecting into a stream or the sea from bank or shore to control shoaling and scour by deflection or redirection of currents and waves. When used in stream environments, a common term used for these devices is spur dike.

This classification may be subdivided with respect to permeability. Impermeable jetties being used to deflect the stream and permeable jetties being used not only to deflect the stream but to permit some flow through the structure to minimize the formation of eddies immediately downstream. Most jetty installations are permeable structures.

Permeable jetties typically take the following forms of construction:

- Palisades -- piles and netting.
- Single and double rows of timber-braced piling.
- Steel or timber jacks.
- Precast concrete, interlocking shapes or hollow blocks.

Impermeable jetties typically take the following forms of construction:

- Guide and spur dikes, earth or rock.
- PCC grouted riprap dikes.
- Single and double lines of sheeting or sheet piling (steel, timber or concrete, framed and braced or on piling).
- Double fence, filled.
- Log or timber cribs, filled.

Impermeable jetties in the form of filled fences and cribs have been used with only limited success. Characteristic performance of these is the development of an eddy current immediately downstream which attacks the bank and often requires secondary protective measures.

Basic principles for permeable jetties are much the same as for retards, the important difference being that they deflect the flow in addition to encouraging deposition. The preceding comment on retards should be considered as related and applicable to jetties when qualified by this basic difference.

Permeable jetties are placed at an angle with the embankment and are more applicable in meandering streams for the purpose of directing or forcing the current away from the embankment, see Figure 873.4A. When the purpose is to deposit material and promote growth, the jetties are considered to have fulfilled their function and are expendable when this occurs.

Figure 873.4A

Thalweg Redirection Using Bendway Weirs

Bendway weirs in conjunction with rock slope protection.

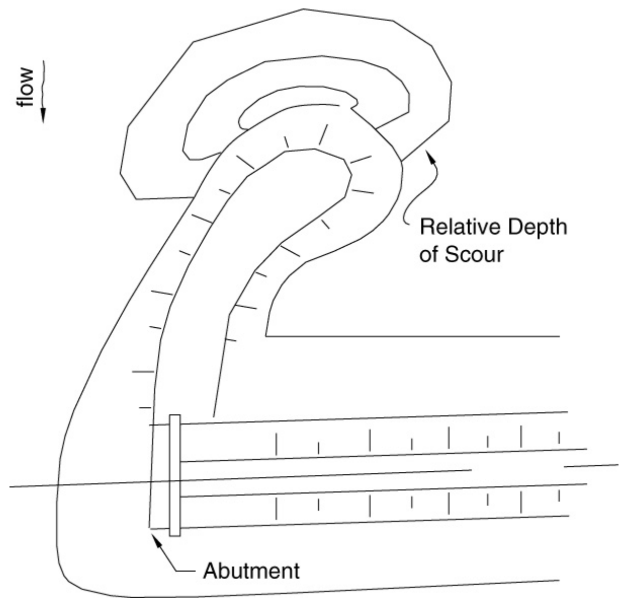
They also encourage deposition of bed material and growth of vegetation. Retards build a narrow strip in front of the embankment, where as permeable jetties cover a wider area roughly limited by the envelope of the outer ends.

The relation between length and spacing of jetties should approximate unity as a general rule to assure complete entrapment and retention of material. The spacing can be increased if the resulting scalloped effect is not detrimental to the desired result. See HEC-23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 9 for additional information.

(c) Guide Dikes/Banks. Guide banks are appendages to the highway embankment at bridge abutments, see Figure 873.4B. They are smooth extensions of the fill slope on the upstream side. Approach embankments are frequently planned to project into wide floodplains, to attain an economic length of bridge. At these locations high water flows can cause damaging eddy currents that scour away abutment foundations and erode approach embankments. The purpose of guide dikes is twofold. The first is to align flow from a wide floodplain toward the bridge opening. The second is to move the damaging eddy currents from the approach roadway embankment to the upstream end of the dike.

Figure 873.4B

Bridge Abutment Guide Banks



Guide banks are usually earthen embankment faced with rock slope protection. Optimum shape and length of guide dikes will be different for each site. Field experience has shown that an elliptical shape with a major to minor axis ratio of 2.5:1 is effective in reducing turbulence. The length is dependent on the ratio of flow diverted from the floodplain to flow in the first 100 feet of waterway under the bridge. If the use of another shape dike, such as a straight dike, is required for practical reasons more scour should be expected at the upstream end of the dike. The bridge end will generally not be immediately threatened should a failure occur at the upstream end of a guide dike.

Toe dikes are sometimes needed downstream of the bridge end to guide flow away from the structure so that redistribution in the floodplain will not cause erosion damage to the embankment due to eddy currents. The shape of toe dikes is of less importance than it is with upstream guide banks.

For further information on spur dike and guide bank design procedures, refer to Section 6.4 of HDS No. 6. General design considerations and guidance for evaluating scour and stream stability at highway bridges is contained in HEC-18, HEC-20, and HEC-23.

(d) Groins. A groin is a relatively slender barrier structure usually aligned to the primary motion of water designed to trap littoral drift, retard bank or shore erosion, or control movement of bed load.

These devices are usually solid; however, upon occasion to control the elevation of sediments they may be constructed with openings. Groins typically take the following forms of construction:

- Rock mound.
- Concreted-rock dike.
- Sand filled plastic coated nylon bags.
- Single or double lines of sheet piling.

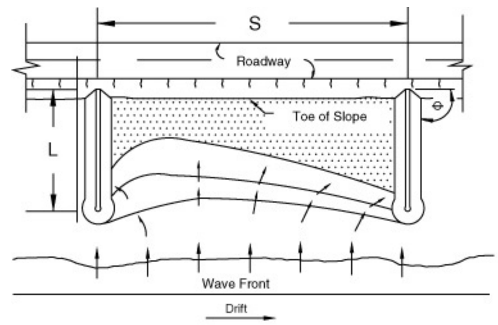
The primary use of groins is for ocean shore protection. When used as stream channel protection to retard bank erosion and to control the movement of streambed material they are normally of lighter construction than that required for shore installation.

In its simplest or basic form, a groin is a spur structure extending outward from the shore over beach and shoal. A typical layout of a shore protection groin installation is shown in Figure 873.4C.

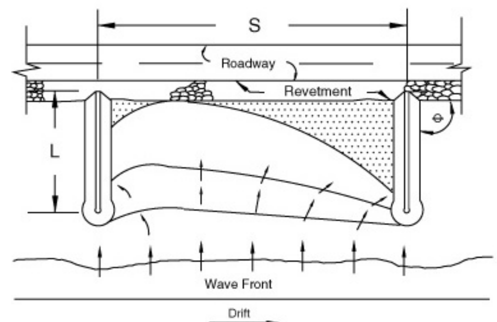
Assistance from the U.S. Army Corp of Engineers is necessary to adequately design a slope protection groin installation. For a more complete discussion on groins, designers should consult Volume II, Chapter 6, Section VI, of the Corps' Shore

Figure 873.4C

Typical Groin Layout With Resultant Beach Configuration



Long Groins Without Revetment



Short Groins With Light Stone Revetment

NOTE:

"S", "L" and "θ" are determined by conditions at site.

Protection Manual until Part VI of the Coastal Engineering Manual is published. Preliminary studies can be made by using basic information and data available from USGS quadrangle sheets, USC & GS navigation charts, hydrographic charts on currents for the Northeast Pacific Ocean and aerial photos of the area.

Factors pertinent to design include:

(1) Alignment. Factors which influence alignment are effectiveness in detaining littoral drift, and self-protection of the groin against damage by wave action.

A field of groins acts as a series of headlands, with beaches between each pair aligned in echelon, that is, extending from outer end of the downdrift groin to an intermediate point on the updrift groin, see Figure 873.4D. The offset in beach line at each groin is a function of spacing of groins, volume of littoral drift, slope of sea bed and strength of the sea, varying measurably with the season. Length and spacing must be complementary to assure continuity of beach in front of a highway embankment.

A series of parallel spurs normal to the beach extending seaward would be correct for a littoral drift alternating upcoast and downcoast in equal measure. However, if drift is predominantly in one direction the median attack by waves contributes materially to the longshore current because of oblique approach. In that case the groin should be more effective if built oblique to the same degree. Such an alignment will warrant shortening of the groin in proportion to the cosine of the obliquity, see Figure 873.4D.

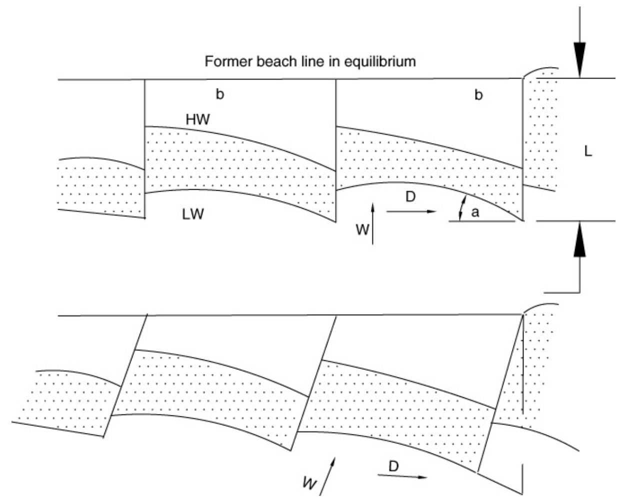
Conformity of groin to direction of approach of the median sea provides an optimum ratio of groin length to spacing, and the groin is least vulnerable to storm damage. Attack on the groin will be longitudinal during a median sea and oblique on either side in other seas.

(2) Grade. The top of groins should be parallel to the existing beach grade. Sand may pass over a low barrier. The top of the groin should be established higher than the existing beach, say 2 feet as a minimum for moderate exposure combined with an abundance of littoral drift, to 5 feet for severe exposure and deficiency of littoral drift.

The shore end should be tapered upward to prevent attack of highway embankment by rip currents, and the seaward end should be tapered downward to match the side slope of the groin in order to diffuse the direct attack of the sea on the end of the groin.

Figure 873.4D

Alignment of Groins to an Oblique Sea Warrants Shortening Proportional to Cosine of Obliquity



(3) Length and Spacing. The length of groin should equal or exceed the sum of the offset in shoreline at each groin plus the width of the beach from low water (LW) to high water (HW) line, see Figure 873.4D. The offset is approximately the product of the groin spacing and the obliquity (in radians) of the entrapped beach. The width of beach is the product of the slope factor and the range in stage. The relation can be formulated:

$$L = ab + rh$$

Where:

L = Length of groin, feet

a = obliquity of entrapped beach in radians

b = beach width between groins, feet

r = reciprocal of beach slope

h = range in stage, feet

For example, with groins 400 feet apart, obliquity up to 20 degrees, on a beach sloping 10:1 with a tidal range of 11 feet,

L = .35 x 400 + 10 x 11 = 250 feet

The same formula would have required L = 390 feet for 800-foot spacing, reducing the aggregate length of groins but increasing the depth of water at the outer ends and the average cost per foot. For some combination of length and spacing the total cost will be a minimum, which should be sought for economical design.

If groins are too short, the attack of the sea will still reach the highway embankment with only some reduction of energy. Some sites may justify a combination of short groins with light revetment to accommodate this remaining energy.

(4) Section. The typical section of a groin is shown in Figure 873.4E. The stone may be specified as a single class, or by designating classes to be used as bed, core, face and cap stones.

Face stone may be chosen one class below the requirement for revetment by Chart A or B, Figure 873.3G. Full mass stone should be specified for bed stones, for the front face at the outer end of the groin, and for cap stones exposed to overrun. Core stones in wide groins may be smaller.

Width of groin at top should be at least 1.5 times the diameter of cap stones, or wider if necessary for operation of equipment. Side slopes should be 1.5:1 for optimum economy and ordinary stability. If this slope demands heavier stone than is available, side slope can be flattened or the cap and face stones bound together with concrete as shown in Figure 873.3F.

(e) Baffle. A baffle is a pier, vane, sill, fence, all or mound built on the bed of a stream to control, deflect, check or disturb the flow or to float on the surface to dampen wave action.

Baffles typically take the following forms of construction:

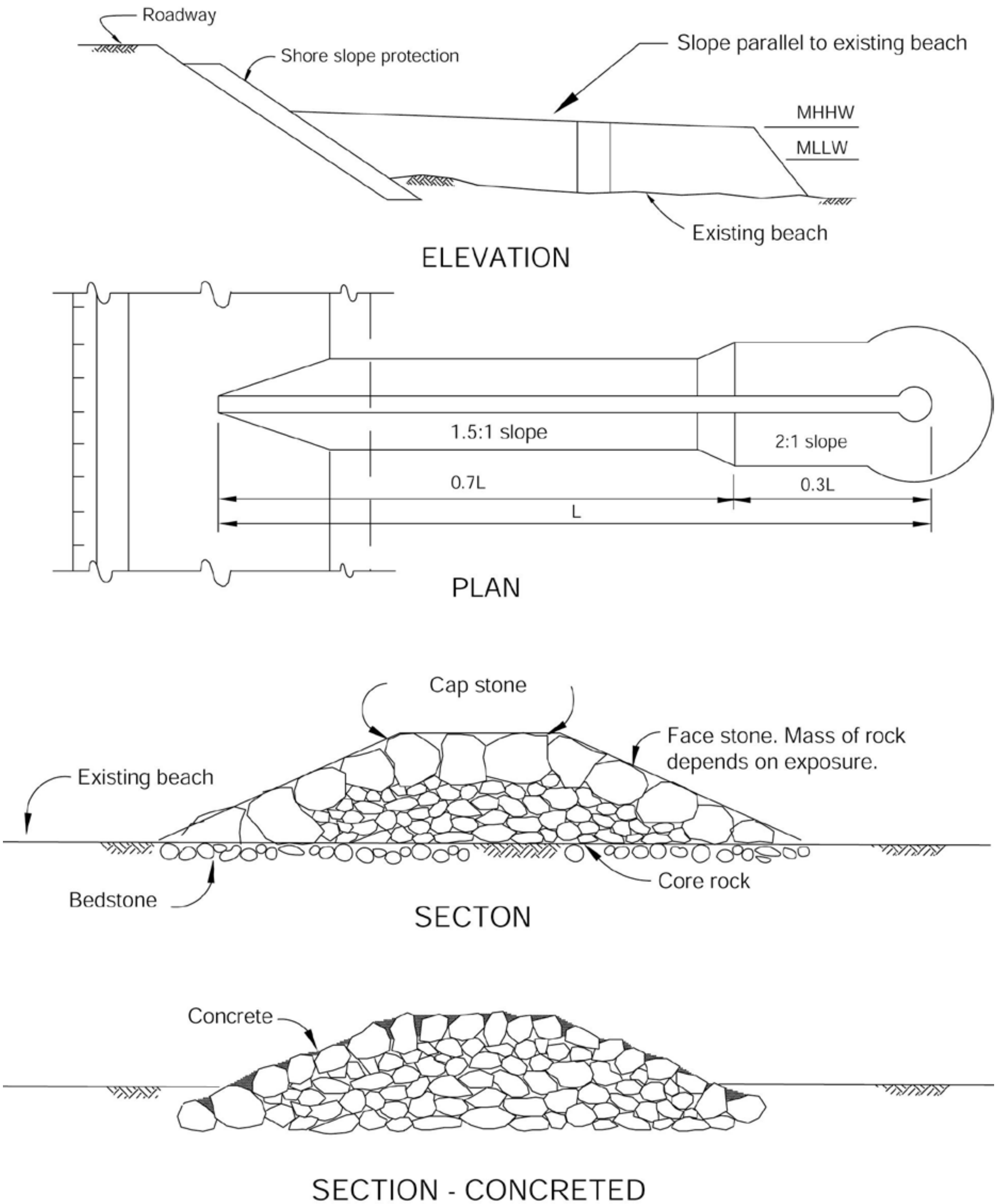
- Single or multiple lines of fence.
- Drop Structures (gabions, rock, concrete, etc.).
- Dikes of earth or rock.
- Floating boom.

These devices may vary in magnitude from a check dam on a small stream to a system of training dikes or permeable jetties for deflecting or directing flow. When using fences, palisades, or dikes as deflectors along the more mature valleys or meandering streams, the potential erosion to previously unexposed areas, threat to adjacent property, eddy currents and possibility of scour should all be assessed. When used as a collecting system to control and direct the flow to new or existing drainage facilities or to bridge openings, the alignment of the installation should be developed as a series of curves and intervening tangents guiding the stream through transitions to maintain smooth and steady flow. The surface and curvature of the training device should be governed by the natural or modified velocity.

Drop structures or check dams are an effective means of gradient control. They may be constructed of rock, gabions, concrete, timber, sacked concrete, filled fences, sheet piling or combinations of any of the above. They are most suited to locations where bed materials are relatively impervious otherwise underflow must be prevented by cutoffs. Refer to HDS No. 6, Section 6.4.11, for further discussion on the use of drop structures.

Floating booms are effective protection against the smaller wave actions common to lakes and tidal basins. Anchorage is the prime structural consideration.

Figure 873.4E
Typical Stone Dike Groin Details



NOTES:

This is not a standard design.

Dimensions and details should be modified as required.

873.5 Design Check List

The designer should anticipate the more significant problems that are likely to occur during the construction and maintenance of channel and shore protection facilities. So far as possible, the design should be adjusted to eliminate or minimize those potential problems.

The logistics of the construction activity such as access to the site, on-site storage of construction materials, time of year restrictions, environmental concerns, and sequence of construction should be carefully considered during the project design. The stream and shoreline morphology and their response to construction activities are an integral part of the planning process. Communication between the designer and those responsible for construction administration as well as maintenance are important.

Channel and shore protection facilities require periodic maintenance inspection and repair. Where practicable, provisions should be made in the facility design to provide access for inspection and maintenance.

The following check list has been prepared for both the designer and reviewer. It will help assure that all necessary information is included in the plans and specifications. It is a comprehensive list for all types of protection. Items pertinent to any particular type can be selected readily and the rest ignored.

1. Location of the planned work with respect to:
   - The highway.
   - The stream or shore.
   - Right of way.
2. Datum control of the work, and relation of that datum to gage datum on streams, and both MSL and MLLW on the shore.
3. A typical cross section indicating dimensions, slopes, arrangement and connections.
4. Quantity of materials (per foot, per protection unit, or per job).
5. Relation of the foundation treatment with respect to the existing ground.
6. Relation of the top of the proposed protection to design high water (historic, with date; or predicted, with frequency).
7. The limits of excavation and backfill as they may affect measurement and payment.
8. Construction details such as weep holes, rock slope protection fabrics, geocomposite drains and associated materials.
9. Location and details of construction joints, cut-off stubs and end returns.
10. Restrictions to the placement of reinforcement.
11. Connections and bracing for framing of timber or steel.
12. Splicing details for timber, pipe, rails and structural shapes.
13. Anchorage details, particularly size, type, location, and method of connection.
14. Size, shape, and special requirements of units such as precast concrete shapes and other manufactured items.
15. Number and arrangement of cables and details of fastening devices.
16. Size, mass per unit area, mesh spacing and fastening details for wire-fabric or geosynthetic materials.
17. On timber pile construction the number of piles per bent, number of bents, length of piling, driving requirements, cut-off elevations, and framing details.
18. On fence-type construction the number of lines or rows of fence, spacing of lines, dimensions of posts, details of bracing and anchorage ties, details of ties at end.
19. The details of gabions and the filling material.
20. The size of articulated blocks, the placement of steel, and construction details relating to fabrication.
21. The corrosion considerations that may dictate specialty concretes, coated reinforcing, or other special requirements.

CHAPTER 880
CURRENTLY NOT IN USE

CHAPTER 890
STORM WATER MANAGEMENT

Topic 891 - General

Index 891.1 - Introduction

The term "storm water management" refers to the cooperative efforts of public agencies and the private sector to mitigate, abate, or reverse the adverse results, both in water quantity and water quality, associated with the altered runoff phenomena that typically accompanies urbanization. Storm water management encompasses a number of control measures, which may be either structural or non-structural (including policy and procedural measures) in nature.

This chapter will focus primarily on the management of storm water runoff quantity. Information related to the designer's responsibility for the management of storm water runoff quality is contained in the Department's Project Planning and Design Guide.

891.2 Philosophy

When runoff impacts result from a Department project, then the cost of mitigating these impacts is a legitimate part of the project cost. Since transportation funds are increasingly limited, and because mitigation of runoff problems can be expensive, it is important to identify the causative factors and responsible parties. When runoff impacts are caused by others, avenues for assigning these costs to the responsible party should be evaluated. The local agencies responsible for land use in the area are a good place to begin this evaluation, as many of these local agencies have enacted land use regulations in an effort to control flooding. These regulations often require that developers limit changes in the volume and rate of discharge between the pre- and post-development site conditions. In addition, many local agencies must be responsive to their own storm water permits which require that they implement programs to control the quality of storm water discharges within their jurisdiction. When runoff impacts are caused jointly by the Department and others, it may be possible to develop cooperative agreements allowing joint impact mitigation. See Indexes 803.2 and 803.3 for further discussion on cooperative agreements and up-grading of existing highway drainage facilities.

Topic 892 - Storm Water Management Strategies

892.1 General

Quantity / Quality Relationship. Management of storm water quality often requires the assessment of relatively small runoff producing events. As much as 80 percent of average annual rainfall is produced by storms with return periods of less than 2 years. As a result, water quality facilities are typically sized to address relatively small runoff volumes. Conversely, storm water quantity management is typically directed at reducing the peak flow rate on storms with a 10-year or greater return period, and water quantity control facilities must be sized accordingly.

In order to achieve both water quantity and quality benefits, it may be necessary to use a combination of strategies or control measures. For example, placement of a relatively small detention basin or filtration immediately upstream of a quantity attenuating detention basin can provide sediment capture, while allowing larger flows to be mitigated by the major basin. Some types of water quality control measures will need to incorporate bypass features so that the smaller, more frequent, runoff events can be treated while still allowing larger flows to be routed away from the traveled way.

892.2 Types of Strategies

There are various storm water management strategies which may be used to mitigate the effects of storm water runoff problems. They vary from very simple to very complex techniques depending upon specific site conditions and regulatory requirements which must be satisfied.

The Department Storm Water Quality Handbook, "Planning and Design Staff Guide" provides both design guidance on specific water quality control measures as well as a more general discussion of how and when to incorporate water quality control measures into projects.

In addition to the measures described in the Storm Water Quality Handbook, the following measures may provide relief in dealing with the water quantity side of storm water management.

(1) *Detention & Retention Basins.* The detention and retention basin designs provided in the Storm Water Quality Handbook are based upon water quality control, not quantity control. Refer to the Department training course manual "Storm Water Management Design" for information related to design considerations for peak flood reduction through the use of detention and retention basins. Also, refer to HEC No.22, Chapter 8.

(2) *Groundwater Recharge.* In some locations highly permeable underground strata may allow percolation of excess runoff into the ground. Benefits include recharge of underground aquifers and the possible reduction or elimination of conveyance systems along with pollutant removal. Special care must be exercised in areas of high groundwater to avoid potential contamination of the aquifer.

(3) *Drainage Easements*. In areas where right of way is inexpensive it may be possible to purchase flood easements. These areas are typically used for agriculture and are subject to flooding at any time during specified times of the year. Cooperative agreements with local agencies or flood control districts will typically be necessary.

892.3 Design Considerations

The items presented below describe some of the issues to be considered prior to, and during, the design of any storm water management facility. General issues common to most storm water management strategies that need to be evaluated are:

- Access for maintenance must be provided, and the facility must be maintainable. Storm water control facilities must not become regarded as wetlands themselves, which would require special permits for routine maintenance.
- Facilities should be designed to "blend in" with their surroundings to the greatest extent possible. The district landscape architecture unit should be contacted for assistance.
- The effects of the proposed facility on channel capacities and existing floodways require evaluation. Care must be taken to evaluate the effects related to the delayed release from detention facilities since an increase in downstream peak discharges may result (see Figure 892.3).
- The effects of releasing sediment free "hungry" water into channels and the potential for increased erosion rates downstream must be determined.
- Evaluate the effects of depriving downstream water users (human, aquatic or vegetative) of runoff due to retention, percolation or other diversion.

Storm water management techniques involving on-site and off-site storage may offer the highway design engineer the more reasonable and responsive solution to problems relative to the handling of excess runoff. The cooperation of other jurisdictions is generally a prerequisite to applying these strategies and a cooperative agreement is almost always necessary. See Chapter 12 of the AASHTO Model Drainage Manual for additional design criteria for storage facilities.

892.4 Mixing with Other Waste Streams

Storm water runoff from State highways will usually be carried to a receiving body of water without being combined with waste water. Although some combined storm and sanitary sewers do exist, their use should be avoided.

The most common areas of waste stream mixing have been at maintenance stations. These facilities may have combined storm water and wash rack systems. Because of wash water and rinse water, maintenance stations present unique water quality problems from concentrated levels of pollutant loadings. The preferable design has a separate system for the wash rack so that it is not mixed with storm water and rinse water. For additional advice on treatment of concentrated waste streams at maintenance stations, contact the Water/Waste Water Unit in the Division of Engineering Services – Structures Design.

Topic 893 - Maintenance Requirements for Storm Water Management Features

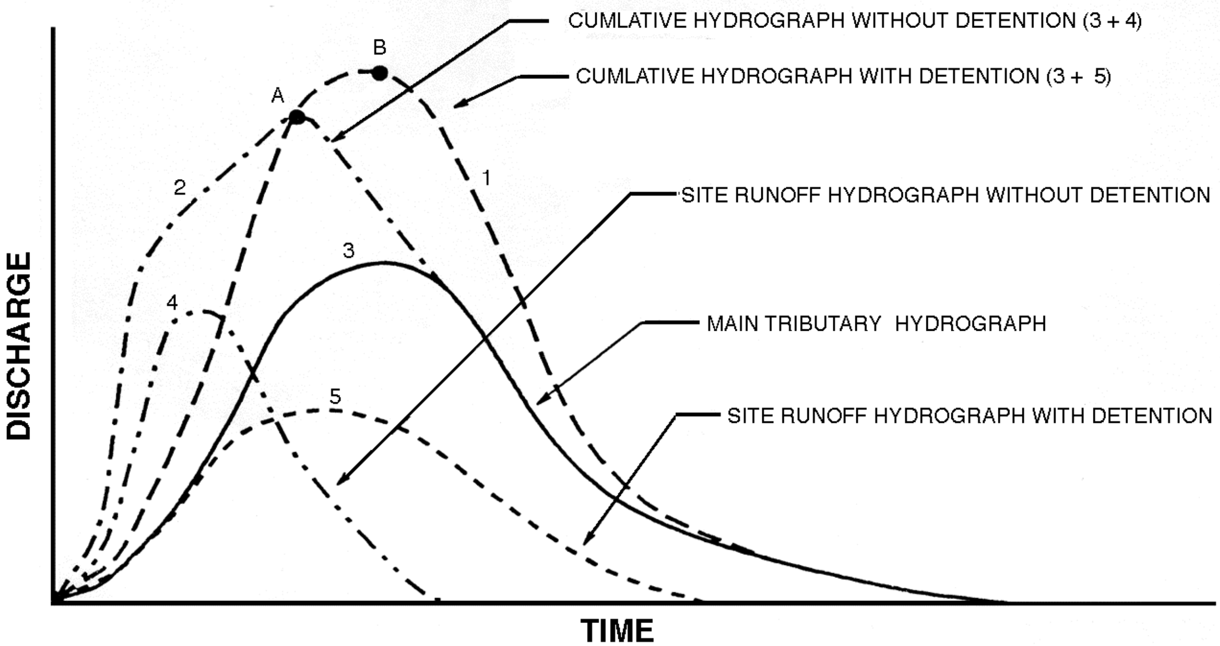
893.1 - General

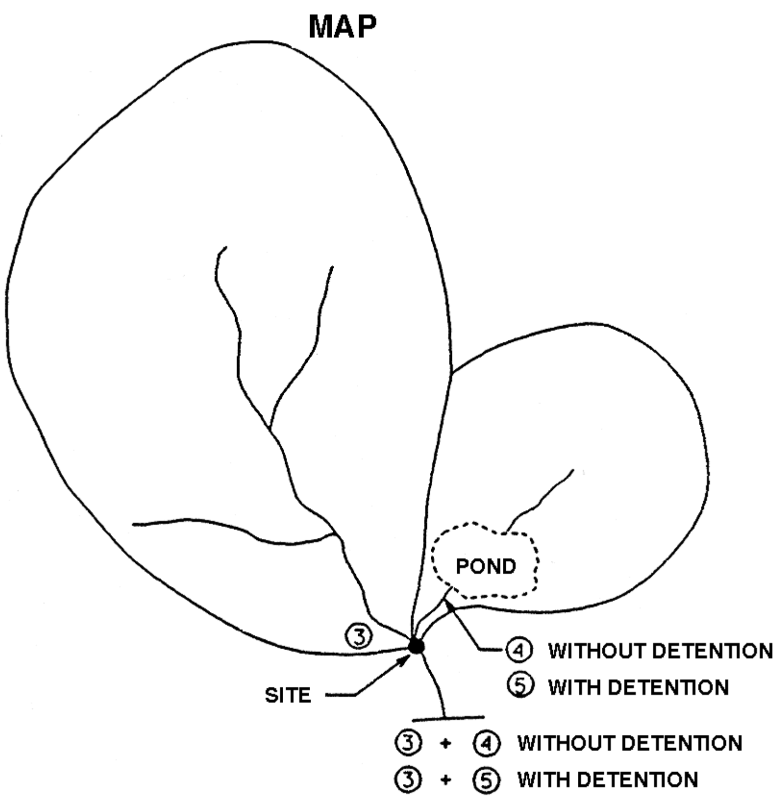
As mentioned previously, the ability and the commitment to maintain storm water management facilities is necessary for their proper operation. The designer must consider the maintenance needs, and the type of maintenance that will take place, in order to provide for adequate access to and within the facility site.

Additionally, the designer should initiate both verbal and written contact with District maintenance to verify the availability of resources to provide proper maintenance and to keep them aware of potential high maintenance items that will be constructed. Initial estimates of how often sediment removal should be performed should be provided by the designer based upon estimated design loadings. Other types of maintenance, such as periodic inspections of embankments, inlet/outlet structures, debris removal, etc. should also be discussed. Due to the large capital investment required for constructing storm water management facilities, proper maintenance cannot be overlooked.

By definition, detained water contributes to runoff and therefore detention ponds or basins must have an outlet and outfall system (see Index 816.4). A gravity outfall should be used whenever feasible. Pumping should only be used where there is no other practical way of handling the excess runoff. See Topic 839 for further discussion on pumping stations.

Figure 892.3
Example of Cumulative Hydrograph With and Without Detention





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 Redwoood (Sequoia sempervirens), Coast Live Oak (Quercus agrifolia) and Deodar Cedar (Cedrus deodora).

<u>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</u>; 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. <u>The minimum tree setback in these cases should be 25 feet.</u>

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	18" Min. from curb face	30' Min from ETW	
With barrier	Min. deflection distance from barrier face (barrier type specific)	Min. deflection distance from barrier face (barrier type specific)	
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 District Design Liaison and Project Delivery 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.

$$\frac{Ramp\ Count}{Mainline\ AADT} = Stopping\ Factor\ (\%)$$

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.

$$Mainline\ AADT\ (Year\ of\ Traffic\ data) \times Stopping\ Factor\ (\%) \times 2.2 = Total\ Visitors\ Per\ Day$$

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.

$$Mainline\ AADT \times Stopping\ Factor\ (\%) \times 2.2 \times 1.8 = Total\ Visitors\ Per\ Day\ (Year\ of\ Traffic\ Data)$$

(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.

$$\frac{Peak\ Hour \times Stopping\ Factor\ (\%) \times 1.8}{3\ Turnovers\ per\ hour} = Total\ Parking\ Spaces\ (Design\ Year)$$

(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 considered at safety roadside rest areas. 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*. <u>The design of connections to vista points should be in accordance with Index 107.1.</u> **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-R2: 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. Design guidance for Class IV bikeways (separated bikeways) is provided in DIB 89. The AASHTO Guide for the Development of Bicycle Facilities also provides additional bikeway guidance not included in this chapter. In addition, bikeway publications and manuals developed by organizations other than FHWA and AASHTO also provide guidance not covered in this manual.

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, III, and cycle tracks or separated 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.

(5) *Class IV Bikeways (Separated Bikeways).* See DIB 89 for guidance.

A Class IV bikeway (separated bikeway) is a bikeway for the exclusive use of bicycles and includes a separation required between the separated bikeway and the through vehicular traffic. The separation may include, but is not limited to, grade separation, flexible posts, inflexible posts, inflexible barriers, or on-street parking. See DIB 89 for further Class IV guidance.

It is emphasized that the designation of bikeways as Class I, II,III, and IV 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 to 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. However, alternating from Class IV to Class II may be appropriate due to the presence of many driveways or turning movements. The highway context or community setting may also influence the need to alternate bikeway classifications.

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. <u>3 feet should be provided.</u> 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.

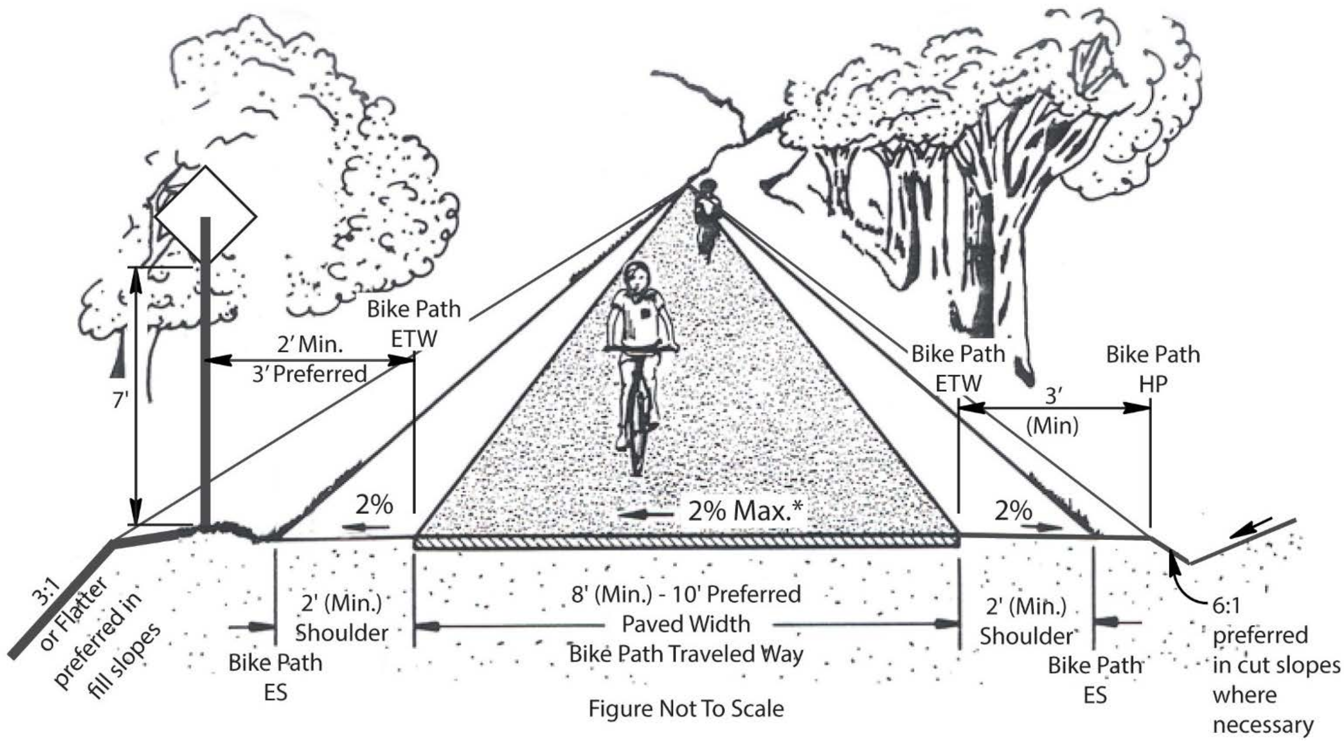
Where motor vehicle cross traffic and bicycle traffic is heavy, grade separations are desirable to eliminate intersection conflicts. Where grade separations are not feasible, assignment of right of way by traffic signals should be considered. Where traffic is not heavy, ”STOP” or “YIELD” signs for either 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

Figure 1003.1A

Two-Way Class I Bikeway (Bike Path)



NOTES:

(1) See Index 1003.1(13) for pavement structure guidance of bike path.

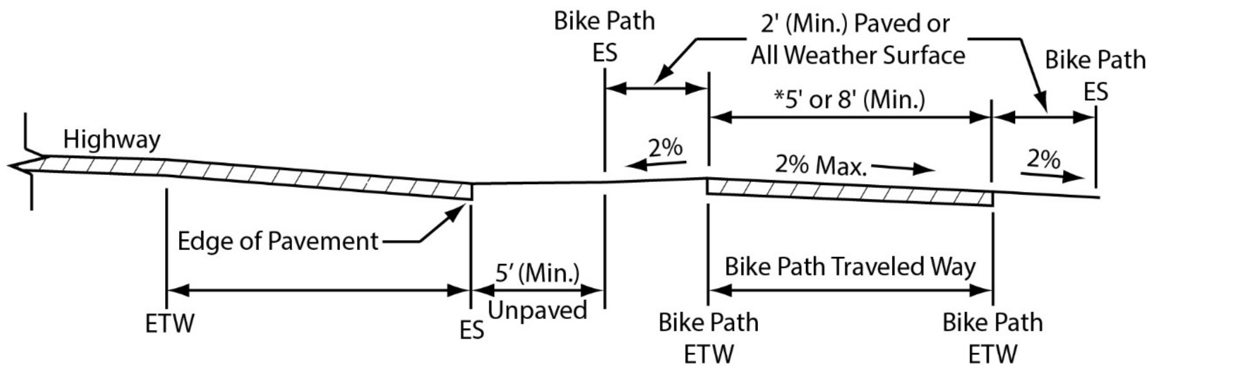
(2) For sign clearances, see California MUTCD, Figure 9B-1.

(3) The AASHTO Guide for the Development of Bicycle Facilities provides detailed guidance for creating a forgiving Class I bikeway environment.

* 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

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 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*. <u>Bike paths should not be placed in the median of a State highway or local road</u>, **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.

Figure 1003.1C

Minimum Length of Bicycle Path Crest Vertical Curve (L) Based on Stopping Sight Distance (S)

$L = 2S - \frac{1600}{A}$ when S > L

$L = \frac{AS^2}{1600}$ when S < L

Double line represents S = L

L = Minimum length of vertical curve – feet

A = Algebraic grade difference - %

S = Stopping sight distance – feet

Refer to Index 1003.1(11) to determine “S”, for a given design speed “V”

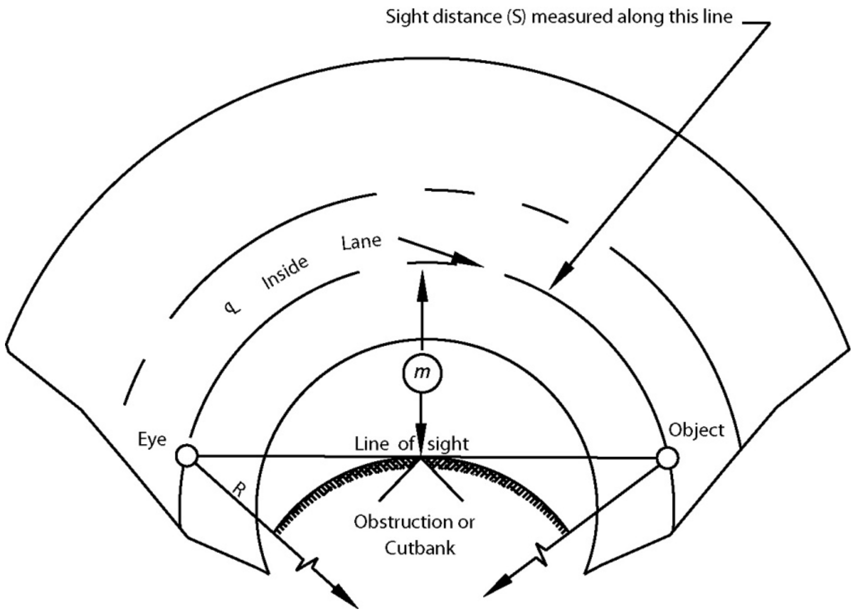
Height of cyclist eye = 4½ feet

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 ℄ of lane in feet.

m = Distance from ℄ of lane in feet.

Refer to Index 1003.1(11) to determine "S" for a given design speed "V".

Angle is expressed in degrees

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

$$\mathrm{S} = \frac{\mathrm{R}}{28.655}\left[\cos^{-1}\left(\frac{\mathrm{R}\text{-}m}{\mathrm{R}}\right)\right]$$

Formula applies only when S is equal to or less than length of curve.

Line of sight is 28" above ℄ inside lane at point of obstruction.

Height of bicyclist's eye is 4 ½ 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

- 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 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.

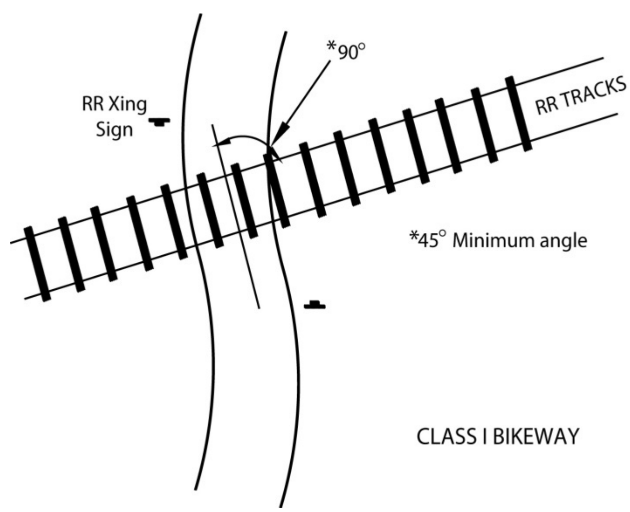
All railroad crossings are regulated by the California Public Utilities Commission (CPUC). All new bicycle path railroad crossings must be approved by the CPUC. Necessary railroad protection will be determined based on a joint field review involving the applicant, the railroad company, and the CPUC.

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.

CHAPTER 1100 HIGHWAY TRAFFIC NOISE ABATEMENT

Topic 1101 - General Requirements

Index 1101.1 - Introduction

The abatement of highway traffic noise is a design consideration that is required by State and Federal Statutes and regulations and by Department policy. This chapter provides design standards relating to the location, height and length of noise barriers and includes discussion on alternative designs, maintenance and emergency access considerations and aesthetics of noise barriers. Procedures and policies on minimum attenuation, design goals, assessing noise impacts, noise abatement criteria levels, priorities, feasibility and reasonableness, and cost-effectiveness are contained in the Project Development Procedures Manual (produced by the Division of Design), the California Traffic Noise Analysis Protocol, and its companion publication, Technical Noise Supplement (both produced by the Division of Environmental Analysis).

1101.2 Objective

The objectives are: for new construction or reconstruction of highways, to limit the intrusion of highway noise into adjacent areas; on existing freeways to limit the noise intrusion to achievable levels within practical and financial limitations; and to limit the noise to the levels specified by statute for qualifying schools adjacent to freeways. To achieve these objectives the Department supports the following four approaches to alleviate traffic noise impacts:

(1) *Reduction at the Source.* Reduction of traffic noise at the source is the most cost effective noise control strategy. Therefore, the Department encourages and supports design measures that reduce traffic noise impacts on adjacent roadside communities.

Designers are encouraged to consider mitigating traffic noise at the tire/pavement interface in order to minimize noise emanating from the highway. Quieter pavement strategies exist for flexible and rigid pavements on and off of structures. Refer to the Quiet Pavement Bulletin dated October 6, 2009 and the Pavement Program for more information. Low noise rumble strips are under development for reducing exterior roadside noise levels while maintaining or increasing interior vehicle noise and tactile feedback.

(2) *Encouraging Compatible Adjacent Land Use.* The Department encourages local governments controlling development or land use near known highway locations to exercise their powers and responsibility to minimize the effect of highway vehicle noise through appropriate land use control. For example, cities and counties have the power to control development by the adoption of land use plans and zoning, subdivision, building and housing regulations.

(3) *Noise Abatement.* The Department will attempt to locate, design, construct, and operate State highways to minimize the intrusion of traffic noise into adjacent areas. When this is not possible, noise impacts may be attenuated by the construction of noise barriers. Construction of noise barriers must result in at least a 5 decibel reduction of noise at the impacted receptors.

(4) *Noise Abatement by Others.* An increasing number of requests are being made to the Department by owners or developers to attenuate noise reaching adjacent properties for which the State's mitigation priority is low or nonexistent. The general policy is that all feasible steps must be taken in the design of the adjacent development to attenuate noise so as not to require encroachment on the State's right of way. The State shall assume NO review authority or responsibility of any kind for the structural integrity or the effectiveness of the sound attenuation of walls constructed by others outside of the State's right of way. Where it is determined to be necessary to permit others to construct a noise barrier within the State's right of way, the general policy is that the design will meet geometric, structural, acoustic, and safety standards as established in this and other manuals and that the effects of the barrier on operation, maintenance and

aesthetics of the highway will be more beneficial than detrimental.

1101.3 Terminology

The terms “noise barrier” and “soundwall” are often used interchangeably. Technically, a “noise barrier” may be any feature which blocks, prevents or diminishes the transmission of noise. An earth berm could serve this purpose. A large building could serve as a noise barrier to shield receptors from the noise source. A dense growth of vegetation, if it were wide enough and dense enough, could be considered a noise barrier. Studies have shown, however, that adequate density would equate to a vegetative expanse of at least 100 feet. A “soundwall” is a particular type of noise barrier. It is a wall, which may be constructed of concrete panels, masonry blocks, wood boards or panels, or a variety of other materials.

1101.4 Procedures for Assessing Noise Impacts

Highway traffic noise impacts are identified in the project noise study report and are listed in the environmental document. The procedures for assessing noise impacts for new highway construction or reconstruction projects, retrofit projects (Community Noise Abatement Program - HB311) along existing freeways, and School Noise Abatement Projects (HB312), are included in Title 23, United States Code of Federal Regulations Part 772, the California Traffic Noise Analysis Protocol, the Project Development Procedures Manual, and Section 216 of the Streets and Highways Code.

1101.5 Prioritizing Construction of Retrofit Noise Barriers

Legal requirements and procedures for prioritizing the construction of noise attenuation barriers are provided in Section 215.5 of the Streets and Highway Code and in the California Traffic Noise Analysis Protocol.

Topic 1102 - Design Criteria

1102.1 General

This section covers the noise barrier location, various design aspects such as height and length of noise barriers, alternative designs, maintenance considerations, and aesthetic considerations. Various types of Department standards and pre-approved alternative noise barrier designs are referenced. Noise barrier design procedures, from the acoustical standpoint, are included in the California Traffic Noise Analysis Protocol. Noise level criteria and guidelines on noise reduction can be found in the California Traffic Noise Analysis Protocol and the Project Development Procedures Manual.

1102.2 Noise Barrier Location

(1) *Lateral Clearances.* **Minimum lateral clearance to noise barriers shall be as provided in Topic 309.1, Horizontal Clearances, of this manual, but shall not be less than 10 feet.** Lateral clearances greater than the minimums should be used whenever feasible. Where terrain permits, the most desirable location for a noise barrier from a safety perspective is just inside the right of way or, alternatively, 30 feet or more from the traveled way.

    When lateral clearance is 15 feet or less, the noise barrier shall be placed on a safety shape concrete barrier. Guardrail or safety shape barrier protection should be considered when the noise barrier is located between 15 feet and 30 feet from the edge of traveled way.

    When the noise barrier is placed closer than 16 feet from the traveled way, Traffic Operations should be consulted early in the design. Signs (overhead and ground mounted) and other poles and standards for lighting, Transportation Management items, call boxes, etc. should be detailed for mounting on the wall, incorporated into the wall foundation and possibly recessed into the surface of the wall.

(2) *Sight Distance Requirements.* The stopping sight distance is of prime importance for noise barriers located on the edge of shoulder along the inside of a curve. Horizontal clearances which reduce the stopping sight distance should be avoided. Noise barriers within gore areas should begin or end at least 200 feet from the theoretical curb nose location.

(3) *Ultimate Location.* Noise barriers should be constructed at the ultimate location -- at the appropriate height and upon the proper

foundation -- for the facility as discussed in the Project Development Procedures Manual and the California Traffic Noise Analysis Protocol.

1102.3 Noise Barrier Height and Position

(1) *Minimum Height.* Noise barriers should have a minimum height of 6 feet (measured from the top of the barrier to the top of the foundation).

(2) *Maximum Height.* Noise barriers should not exceed 14 feet in height (measured from the pavement surface at the face of the safety-shape barrier) when located 15 feet or less from the edge of the traveled way, and should not exceed 16 feet in height above the ground line when located more than 15 feet from the traveled way.

(3) *Truck Exhaust Intercept.* Current FHWA noise barrier design procedures result in noise barrier heights which often do not intercept noise emitted from the exhaust stack of trucks. For design purposes, the noise barrier should intercept the line of sight from the exhaust stack of a truck to the receptor. The truck stack height is assumed to be 11.5 feet above the pavement. The receptor is assumed to be 5 feet above the ground and located 5 feet from the living unit nearest the roadway. If this location is not representative of potential outdoor activities, then another appropriate location should be justified in the noise study report.

(4) *Multi-story Development.* The noise barrier should not be designed to shield more than the first story of multi-story residences unless it provides a minimum reduction of 5 decibels for a substantial number of residences at a reasonable increase in cost. If the noise barrier is extended in height to provide attenuation beyond the first story, attenuation should effectively reduce noise by at least 5 decibels at the receptors precipitating the increase in height.

(5) *Parallel Noise Barriers*. Frequently, noise barriers are constructed to shield noise receivers on both sides of a highway. These are referred to as parallel barriers. If the barrier surfaces are hard, relatively smooth, and nonporous, such as concrete or masonry surfaces, the barriers can reflect noise back and forth between the barriers, decreasing their effectiveness. As a result of research performed by the Department and others, reflective parallel barriers should have a width-to-height ratio (W:H) of at least 10:1 to avoid the risk of perceptible reduction in performance of both noise barriers. The width is the distance between the two barriers, and the height is the average height of both barriers with reference to the roadway elevation. For example, two parallel barriers, one 10 feet, the other 14 feet high, should be separated by at least 120 feet to avoid a noticeable degradation in performance. A perceptible, or noticeable decrease in performance is defined as a reduction of 3 decibels or more in noise attenuation.

(6) *Potential Reflection.* Reflected noise may be an issue for elevated receptors on the opposite side of the roadway. Paving to the base of the noise barrier can create a 'hard' surface and in combination with a soundwall can form a concave shape which might focus sound energy on an opposite roadside community. When possible, keep the finish grade to the base of the noise barrier composed of less-reflective 'soft' material such as uncompacted dirt or ground vegetation. Parallel barriers (discussed above) may also raise reflected noise concerns. Traffic variation and metrological influences make noise measurements at large distances imprecise, while extensive noise studies in the past are inconclusive at finding any distinguishable or discernable change in acoustics due to reflection only. To address concerns and/or complaints regarding reflected noise, a number of absorptive noise barrier systems have been pre-approved for use both on and off of structures. The list of pre-approved absorptive noise barrier systems is available on the Division of Engineering Services Authorized Materials List at: http://www.dot.ca.gov/hq/esc/approved_products_list/ .

1102.4 Noise Barrier Length

(1) *General.* Careful attention should be given to the length of a noise barrier to assure that it provides adequate attenuation for the end dwelling. The California Traffic Noise Analysis Protocol provides guidance on determining how far beyond the end dwelling a

noise barrier should be extended. When appropriate, consideration should be given to terminating the noise barrier with a section of the barrier perpendicular to the freeway. This could reduce the overall barrier length, but may require an easement or acquisition from the property owner to permit construction of the noise barrier off the right of way.

(2) *Gap Closures.* In some cases, short gaps may exist between areas qualifying for a noise barrier. The closure of these gaps should be considered on a project by project basis and be justified in the Project Report.

(3) *Local Street Connections.* At on- and off-ramp connections to local streets, the Department's responsibility for noise abatement should be limited to areas where the traffic noise level from the State highway is the predominant noise source.

(4) *Barrier Overlaps*. When the noise barrier has overlapping sections, such as when concealing an access opening, the walls must be overlapped a minimum of 2.5 to 3 times the offset distance in order to maintain the integrity of the sound attenuation.

1102.5 Alternative Noise Barrier Designs

(1) *General.* Every noise barrier that is constructed as a part of new highway construction or reconstruction, or along freeways as a part of the Community and School Noise Abatement Programs, requires at least two alternative designs included in the bid package. Bridge Reference Specifications 51-561(51SWAL), located on the Division of Engineering Services (DES) website provides the means to include alternative soundwall systems in the bid package. The contract plans should include masonry block as the state design and at least one of the approved soundwall systems listed in the Specification 51-56 (51SWAL). An aesthetic features sheet should be included in the plans for both the masonry block soundwall and for each of the alternatives selected.

The masonry block soundwall sheets (B15-1 to B15-15) can be found in the Standard Plans.

Other design alternatives may be considered provided they meet the structural and noise attenuation criteria. Questions regarding the approval status of various designs or products should be directed to the Division of Design, Office of Special Projects.

Project Files for each noise barrier project should include the justification and background for the design type or the options allowed on each project.

(2) *Design Procedures.* As a minimum, the soundwall plans are to show each of the following:

- Horizontal alignment
- Wall profile made up of a top of Soundwall line and a Top of Footing/Concrete, Barrier/Retaining Wall line
- Applicable standard soundwall detail sheets
- Pile spacing
- Footing steps
- Locations of expansion joints
- Access gates
- Aesthetic features sheet

The following guidance should also be used:

- If the profile grade of the soundwall exceeds six (6) percent, the Top of the Soundwall line should be stepped.
- If the soundwall is on a footing and the Top of Soundwall line is stepped, the Top of Footing line should also be stepped.
- If the Top of Soundwall line is parallel to the profile grade, the Top of Footing line should be parallel to the profile grade of the soundwall.
- If the soundwall is on a concrete barrier, the Top of Concrete Barrier line must be constant height above the profile grade and the Bottom of Concrete Barrier line should be shown on the plans.
- If the soundwall is on a Retaining Wall, the Top of Retaining Wall line or the Bottom

of Footing line and Retaining Wall height should be shown on the plans.

- The original ground (OG) line and any known utilities should be shown on the Soundwall Plan sheets.

(3) *Pay Quantities.* Soundwalls are to be measured by the square foot between the elevation lines shown on the plans and the length of the wall. Soundwall footings are to be paid as minor concrete and concrete barriers are to be paid for as concrete barrier (modified). Piles are to be paid for separately to facilitate minor changes in the field.

Refer to the Standard Special Provisions for more information on measurement and pay quantities.

When calculating costs for determining "reasonableness," all pay quantities associated with the proposed soundwalls should be included in the analysis. Refer to the California Traffic Noise Analysis Protocol for a discussion on this topic.

(4) *Working Drawings.* Working Drawings are no longer required for state designed masonry block soundwalls in view of the fact that all the information necessary to construct the wall should be shown in the contract plans. The Special Provisions for Alternative Soundwall systems should require the successful bidder to submit four (4) sets of drawings for initial review and between six (6) and twelve (12) additional sets, as requested by the Engineer, for final approval and use during construction. Refer to Bridge Reference Specification 51-561(51SWAL) for more information.

(5) *Preliminary Site Data.* In using the "Top of Soundwall/Bottom of Concrete Barrier" line concept, it is important that the preliminary site data be as complete as possible. To eliminate or minimize construction change orders the following guidance is provided:

- Provide accurate ground line profiles.
- Select only standard or pre-approved design alternative soundwall types.
- Provide adequate information based on foundation investigation.
- Locate overhead and underground utilities.
- Review drainage and show any modifications on the plans.
- Determine and specify architectural treatment.
- Determine the need for special design, and coordinate with the Office of Structures Design during the early stages of design.

1102.6 Noise Barrier Aesthetics

(1) *General.* A landscaped earth berm or a combination wall and berm tend to minimize the apparent noise barrier height and are an aesthetically acceptable alternative among noise barrier options; however, these alternatives are not always suitable for many sites due to limited space.

Some additional cost to enhance the aesthetic quality of the noise barrier is usually warranted. Early community involvement toward proposing asesthetic treatment improvements on noise barriers is recommended to accommodate contextual considerations. However, accountability for designs that significantly increase the cost of the noise barrier should be a topic for discussion early in the design process.

Soundwalls should not be designed with abrupt beginnings or ends. Generally, the ends of the soundwall should be tapered or stepped if the height of the soundwall exceeds 6 feet. See Standard Plans for further details. Consult the District Landscape Architect regarding the design of tapers or stepped ends, aesthetic treatment, highway planting and landscaping adjacent to noise barriers.

(2) *Aesthetic Treatment.* Standard aesthetic treatments have been developed by the DES Office of Structure Design for the various alternative materials.

When treatment that is not a standard aesthetic treatment is proposed for noise barriers, contact the District Landscape Architect for selection of the most appropriate treatment. The Headquarters Traffic Liaison should be consulted in these instances to ensure that the treatment of choice satisfies all safety requirements.

(3) *Planting Near Noise Barriers*. The use of plants in conjunction with noise barriers can help to combat graffiti and promote public acceptance of the noise barrier. When landscaping is to be placed adjacent to the soundwall, which will eventually screen a substantial portion of the wall, only minimal aesthetic treatment is justified.

See Index 902.3 and the Project Development Procedures Manual for additional information.

(4) *Transparent Barriers.* Noise barriers may impact viewsheds where consideration of transparent barriers may be warranted. A list of pre-qualified transparent barrier systems is available on the new products list at: www.dot.ca.gov/hq/esc/approved_products_list/.

1102.7 Maintenance Consideration in Noise Barrier Design

(1) *General.* Noise barriers placed within the area between the shoulder and right of way line complicate the ongoing maintenance operations. When there is a substantial distance behind the noise barriers and in front of the right of way line, special consideration is required. If the adjoining land is occupied with streets, roads, parks, or other large parcels, an effort should be made during the right of way negotiations to have the abutting property owners maintain the area. In this case, the chain link fence at the right of way line would not be required. Maintenance by others may not be practical if a number of small individual properties abut the noise barrier.

(2) *Access Requirements.* Access to the back side of the noise barrier must be provided if the area is to be maintained by the Department. In subdivided areas, access can be via local streets, when available. If access is not available via local streets, access gates or openings are essential at intervals along the noise barrier. Access may be provided via offsets in the barrier. Offset barriers must be overlapped a minimum of 2.5 to 3 times the offset distance in order to maintain the integrity of the sound attenuation of the main barrier. Location of the access openings must be coordinated with the District maintenance office.

(3) *Noise Barrier Material.* The alternative materials selected for the noise barrier should be appropriate for the environment in which it is placed. For walls that are located at or near the edge of shoulder, the portion of the noise barrier located above the safety-shape concrete barrier should be capable of withstanding the force of an occasional vehicle which may ride up above the top of the safety barrier.

1102.8 Emergency Access Considerations in Noise Barrier Design

(1) *General.* In addition to access gates being constructed in noise barriers to satisfy the Department's maintenance needs, they may also be constructed to provide a means to access the freeway in the event of a catastrophic event which makes the freeway impassable for emergency vehicles. These gates are not intended to be used as an alternate means of emergency access to adjacent neighborhoods. Access to those areas should be planned and provided from the local street system. Small openings may also be provided in the noise barrier which would allow a fire hose to be passed through it. Local emergency response agencies should be contacted early in the design process to determine the need for emergency access gates and fire hose openings.

(2) *Emergency Access Gate Requirements.* Access gates in noise barriers should be kept to a minimum and should be at least 1,000 feet apart. Locations of access should be coordinated with the District Maintenance office. Only one opening should be provided at locations where there is a need for access openings to serve both the emergency response agency and the Department's maintenance forces. Gates should be designed to comply with the soundwall details developed by the Office of Structures Design.

(3) *Fire Hose Access Openings.* When there is no other means of providing fire protection to the freeway, small openings for fire hoses may be provided. Fire hose access should be located as close as possible to the fire hydrants on the

local street system. Where possible, fire hose access should be combined with emergency or maintenance access openings. The Office of Structures Design should be requested to design fire hose access openings.

1102.9 Drainage Openings in Noise Barrier

Drainage through noise barriers is sometimes required for various site conditions. Depending on the size and spacing, small, unshielded openings at ground level can be provided in the barriers to allow drainage and not adversely impact the noise attenuation of the barrier. The following sizes of unshielded openings at ground level are allowed for this purpose:

(a) Openings of 8" x 8" or smaller, if the openings are spaced at least 10 feet on center.

(b) Openings of 8" x 16" or smaller, if the openings are spaced at least 20 feet on center, and the noise receiver is at least 10 feet from the nearest opening.

The location and size of the drainage openings need to be designed based on the hydraulics of the area. The design should take into consideration possible erosion problems that may occur at the drainage openings.

Where drainage requirements dictate openings that do not conform to the above limitations, shielding of the opening will be necessary to uphold the noise attenuation of the barrier. The shielding designed must consider the hydraulic characteristics of the site. When shielding is determined to be necessary, consultation with the District Hydraulics Unit and the HQ Traffic Liaison is recommended, as well as the Division of Environmental Analysis.

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