CALIFORNIA DEPARTMENT OF TRANSPORTATION

MANUAL CHANGE TRANSMITTAL - ERRATA		NO.
TITLE DIVISION OF DESIGN HIGHWAY DESIGN MANUAL ERRATA 07/24/09 TO SIXTH EDITION	APPROVED BY Keith A. Robinson, for TERRY L. ABBOTT, Chief	Date Issued: 07/24/09 Page 1 of 2
SUBJECT AREA Forward; Table of Contents; List of Figures; List of Tables; Chapters: 60, 80, 100, 200, 300, 400, 500, 600, 700, 800, 900, and 1000	ISSUING UNIT DIVISION OF DESIGN	
SUPERCEDES SEE BELOW FOR SPECIFIC PAGE NUMBERS	DISTRIBUTION **ALL HOLDERS OF THE SIXTH EDITION, HIGHWAY DESIGN MANUAL**	

The California Department of Transportation has identified several typographical errors in the Sixth Edition, Highway Design Manual (HDM). Corrections to the Forward, Table of Contents; List of Figures; List of Tables; Chapters: 60, 80, 100, 200, 300, 400, 500, 600, 700, 800, 900, and 1000 are contained in the errata sheets available on the Department Design website at: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm. Corrections were also made throughout the manual where the new title of the California Manual on Uniform Traffic Control Devices (California MUTCD) is referenced.

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

A summary of substantive typographical corrections are as follows:

Forward

Second paragraph was corrected to include external agencies who use or choose to adopt this guidance. "Organization of the Manual" section contained the incorrect chapter and heading references for Federal-Aid and Federal-Aid Systems.

Figure 201.4

Stopping Sight Distance on Crest Vertical Curves, Page 200-5
The dimension for drivers eye height for stopping sight distance is incorrect. The correct driver eye height is 3 ½ ft. The S=L line plotted on the graph was corrected.

Figure 201.6

Stopping Sight Distance on Horizontal Curves, Page 200-7
Design speeds should be indicated along the top margin of the graph, and thus corrected.

Figure 201.7

Decision Sight Distance on Crest Vertical Curves, Page 200-8
The S=L line plotted on the graph was corrected.

Figure 202.5A

Superelevation Transition, Page 200-13
The columns of data at the bottom of the figure are incorrectly shifted to the left of their headings. All data was realigned beneath its corresponding heading.

Index 206.3(2)

Pavement Reductions, Page 200-28
Second sentence – Referenced Figure 504.3C was corrected to Figure 504.2L.

Figure 307.4	**Geometric Cross Sections for Freeways and Expressways, Page 300-18** Note box, middle of figure – For curbs, referenced Index 504.3(10) was corrected to Index 504.3(11).
Index 404.1(2)	**Corner Sight Distance, Page 400-17** Subsection (a) General, fourth paragraph, mandatory standard – Set back for the driver on the crossroad was incorrectly given as not less than 13 feet rather than 15 feet.
Index 405.3(3)	**Right-turn Lanes at Off-ramp Intersections, Page 400-25** Second paragraph – Referenced Index 504.3(2) was corrected to Index 504.3(3).
Figure 504.2A	**Single Lane Freeway Entrance, Page 500-11** Note 1 pertaining to where right paved shoulder width shall be 10 ft is incorrectly shown within the acceleration lane portion of the figure at the top of the page. Circled reference Note 1 was corrected to the right of the inlet nose on the figure.
Index 504.1(3)	**Location on a Curve, Page 500-13** Second paragraph, last sentence – Corrected reference to note 3 rather than note 5.
Table 504.3A	**Ramp Widening for Trucks, Page 500-15** Lane width for ramp radii less than 150 feet should be 17 ft not 18 ft.
Figure 504.3C	**Typical Freeway Entrance Loop Ramp With 2-Lane Ramp Meter, Page 500-24** Note 4, second paragraph – Reference to Figure 504.3D was corrected to 504.3E.
Figure 504.3J	**Location of Ramp Intersections on the Crossroads, Page 500-31** The notes at the bottom of the Figure were corrected to be consistent with the previous correction in Index 404.1(2).
Figure 504.7A	**Design Curve for Freeway and Collector Weaving, Page 500-40** Example text at bottom of figure incorrectly noted weaving distance L=400m rather than 1300 ft.
Index 603.3(2)	**Capital Preventive Maintenance (CAPM), Page 600-5** First bullet – The thickness for surface course overlays was incorrectly given as less than or equal to 0.25 ft rather than 0.20 ft.
Index 604.2(5)	**Department Pavement website, Page 600-9** The website address was corrected.
Table 622.1	**Rigid Pavement Engineering Properties, Page 620-4** The value for thermal conductivity was corrected to 1.25 Btu/hr-ft-°F.
Figure 623.1M	**Rigid Pavement Catalog (High Mountain/Low Mountain, Type II Subgrade Soil), Page 620-20** The title was corrected to "(High Mountain/High Desert, Type II Subgrade Soil)".
Table 663.1B	**Gravel Factor and California R-values for Bases and Subbases, Page 660-4** The gravel factor equation for Lime Treated Subbase was corrected to 0.9+(USC/1,000).
Index 866.2(3)	**Superelevation, Page 860-12** Last bullet, Trapezoidal Channels – The equation for Supercritical flow was corrected by deleting the multiple of 2 and parenthesis in the denominator.

Enclosures available on the Department Design website at: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm.

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FOREWORD

Purpose

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

The policies established herein are for the information and guidance of the officers and employees of the Department, as well as external agencies who use or choose to adopt this guidance.

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

It is not intended that any standard of conduct or duty toward the public shall be created or imposed by the publication of the 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 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. All projects designed and constructed in English units shall follow the standards in this manual per the instructions contained in Index 82.5, “Effective Date for Implementing Revisions to Design Standards”.

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 be delivered in Metric units. See Department memorandum dated June 16, 2006, signed by Richard D. Land, entitled “Declaration of Units of Measure – “Metric” or “English” Project.”

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 ($kg \cdot m/s^2$)
Energy/work/quantity of heat	joule	J($N \cdot m$)
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

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 (CM301). 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 due to thermal and moisture variations, consolidation, traffic action, or reflections from an underlying pavement.

(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).* Summation of equivalent 18-kip single axle loads used to convert mixed traffic volume to total accumulated traffic loading during the design life of the pavement.

(21) *Flexible Pavement.* Pavements engineered to transmit and distribute traffic 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 a base and a subbase. These types of pavements are called "flexible" because the total pavement structure bends or flexes to accommodate deflection bending under traffic 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 Gf 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 traffic 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 traffic 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 its terminal serviceability or a condition that requires major rehabilitation or reconstruction. 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.

(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

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 for safe and efficient transportation.

(b) Attainment of community goals and objectives.

(c) Needs of low mobility and disadvantaged groups.

(d) Costs 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 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 different transportation modes working together effectively.

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

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 shall

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 are categorized in order of importance in development of a safe State highway system operating at selected levels of service commensurate with projected traffic volumes and highway classification.

(2) *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 below). Mandatory standards use the word "shall" and are printed in **Boldface** type (see Table 82.1A).

(3) *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 Underlining (see Table 82.1B).

(4) *Permissive Standards*. All standards other than mandatory or advisory, whether indicated by the use of "should" or "may", are permissive with no requirement for application intended.

(5) *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).

(6) *Other.* In addition to the design standards in this manual, the Traffic Manual contains standards relating to clearzone, signs, delineation, barrier systems, signals, and lighting.

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.

82.2 Approvals for Nonstandard Design

(1) *Mandatory Standards*. **To promote uniform practice on a statewide basis, design features or elements which deviate from most mandatory standards indicated herein shall require the approval of the Chief, Division of Design. This approval authority has been delegated to the Design Coordinators, except the mandatory standards in Chapters 600 through 670, which have been delegated to the Chief, Office of Pavement Design, and may involve coordination with the Design Coordinator.**

The current procedures and documentation requirements pertaining to the approval process for those exceptions to mandatory design standards that have been delegated to the Design Coordinators are contained in Chapter 21 of the Project Development Procedures Manual (PDPM).

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

A detailed plan for moving traffic 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, 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 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 good 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 down.

The traffic control plans should be consistent with the California MUTCD, and the philosophies and requirements contained in standard lane closure 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 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.
- Consideration for bicycle, pedestrian, and ADA (see Index 105.3) traffic.
- 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 to be performed, and the traveled way to be used by all movements 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 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 HQ, Traffic Operational Systems Branch, 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 prosecute 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 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

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 driver. Four types of sight distance are considered here: 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 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 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 standards for stopping sight distance related to design speed, and these shall be the minimum values used in design. Also shown are the values for use in providing passing sight distance.

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)
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 driver of a vehicle, traveling at a given speed, to bring the vehicle to a stop after an object on the road becomes visible. Stopping sight distance 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(9) for bicycle 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 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 Design Coordinator 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.

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.)

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. 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

Maximum superelevation rates for various highway conditions are shown on 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 mph to approximately 0.35 at 60 mph." 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).

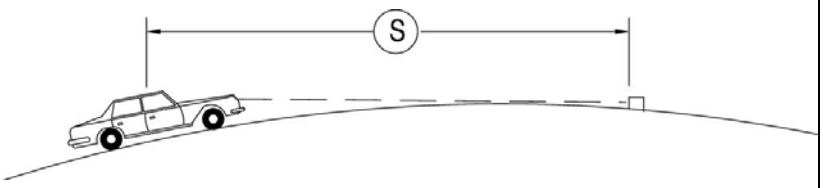
<u>On rural 2-lane roads, superelevation should be on the same plane for the full width of traveled way and shoulders, except on transitions (see Index 304.3 for cut widening conditions).</u>

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.

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 Design Reviewer and/or the Design Coordinator to determine the proper solution and the necessity of preparing a design exception fact sheet. In warping street or ramp surface areas for

Figure 201.4
Stopping Sight Distance on Crest Vertical Curves



Drivers eye height is 3 ½ 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

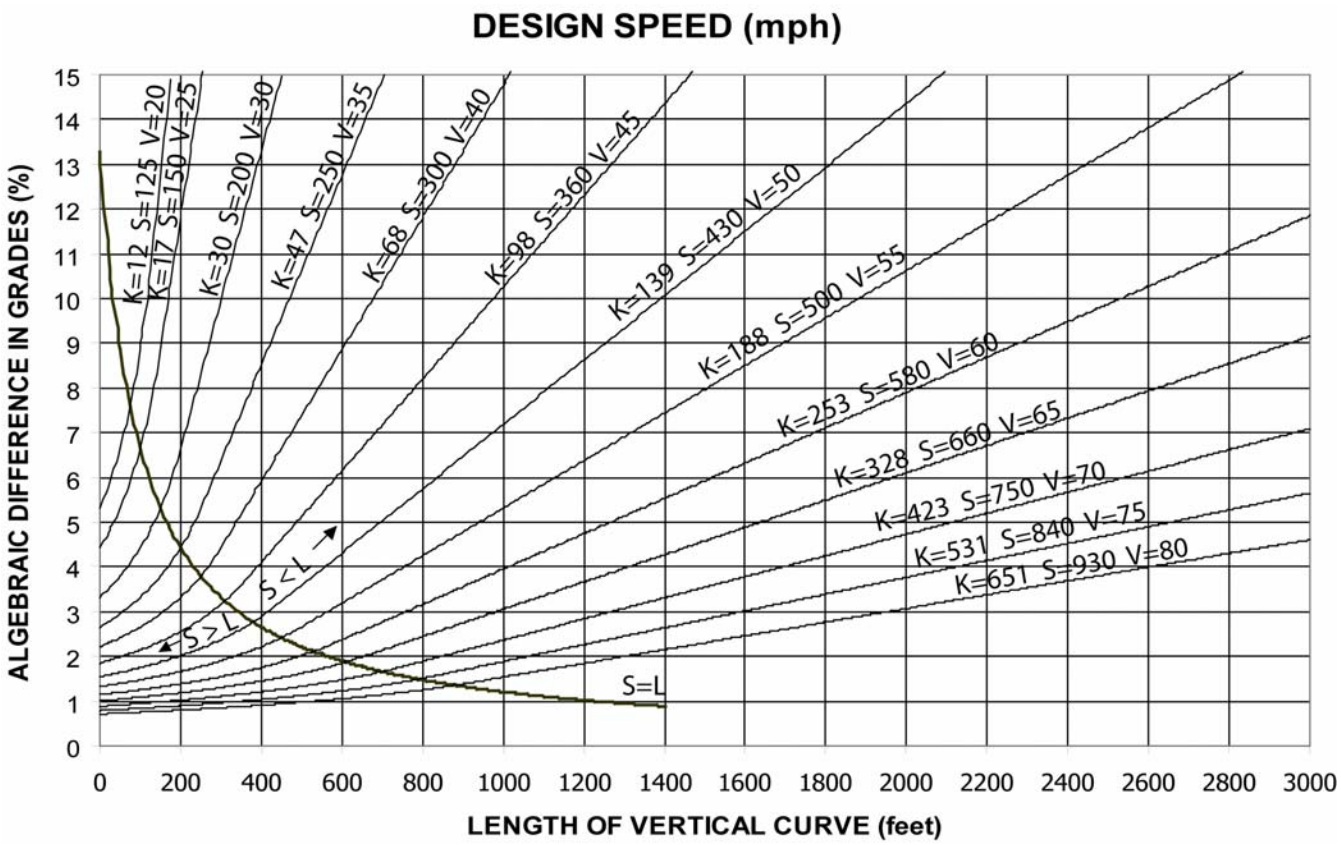
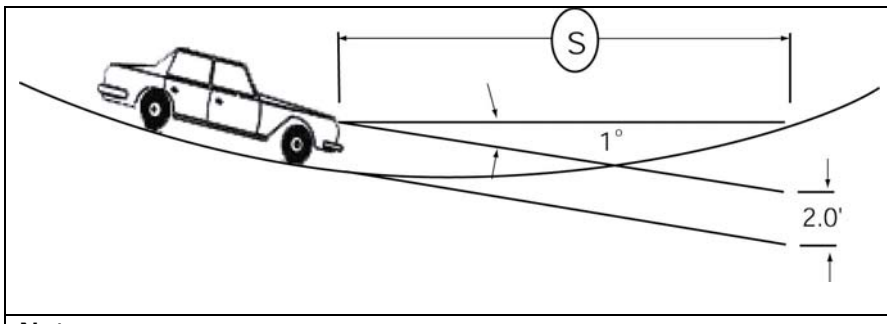


Figure 201.5
Stopping Sight Distance on Sag Vertical Curves



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)

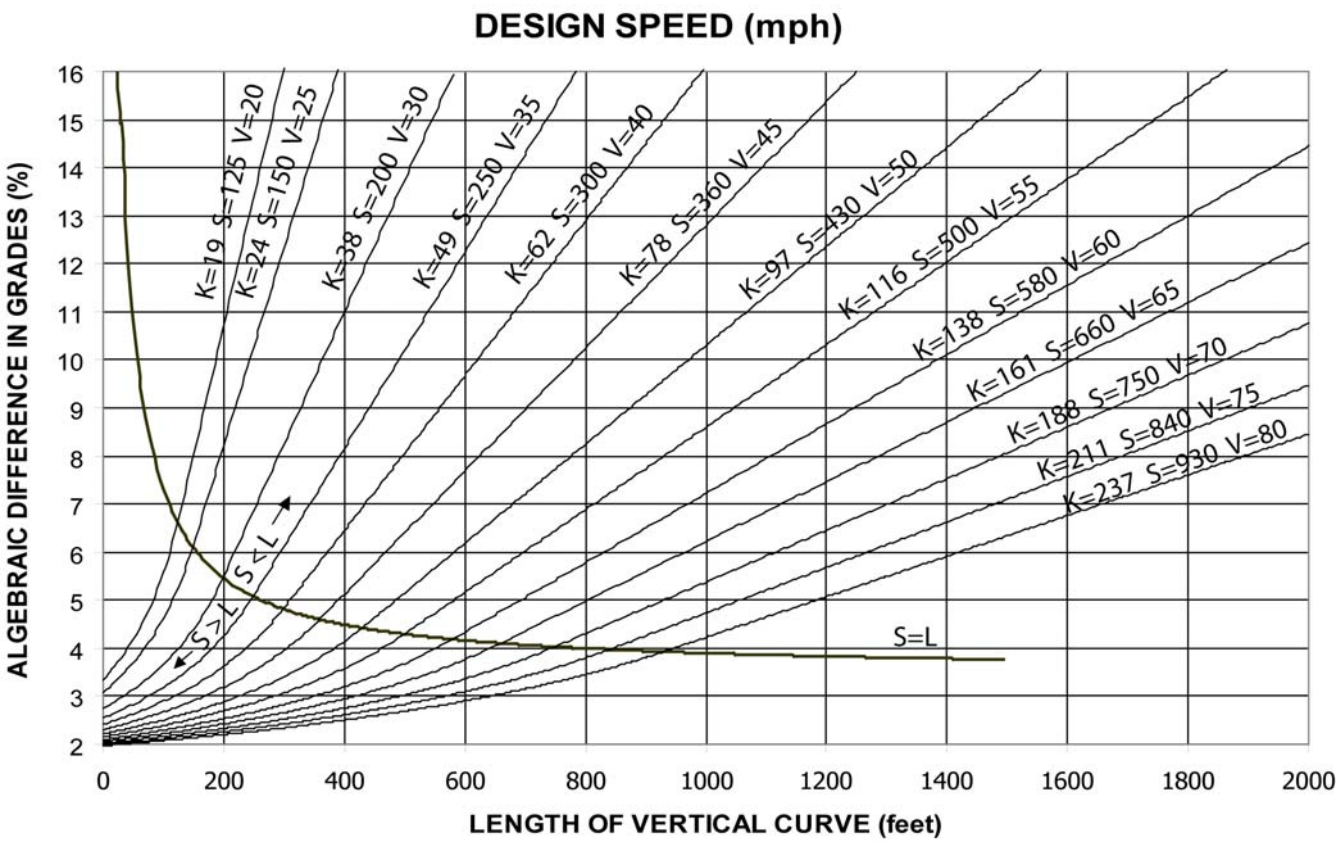
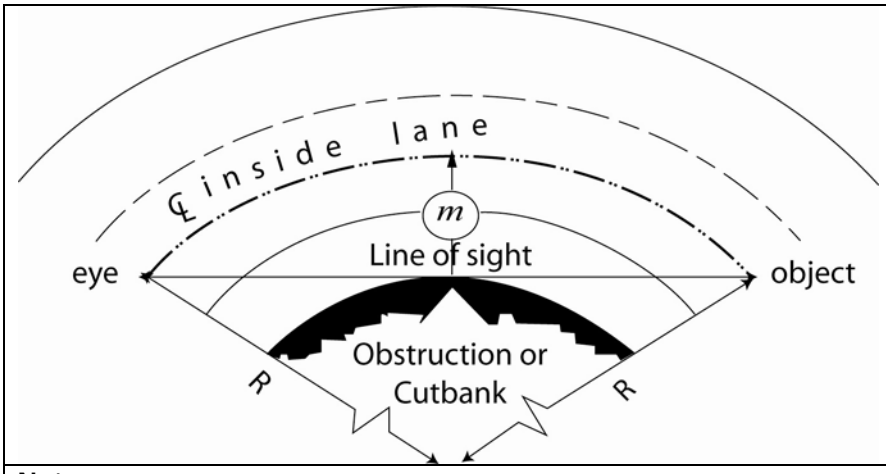


Figure 201.6
Stopping Sight Distance on Horizontal Curves



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.

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

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

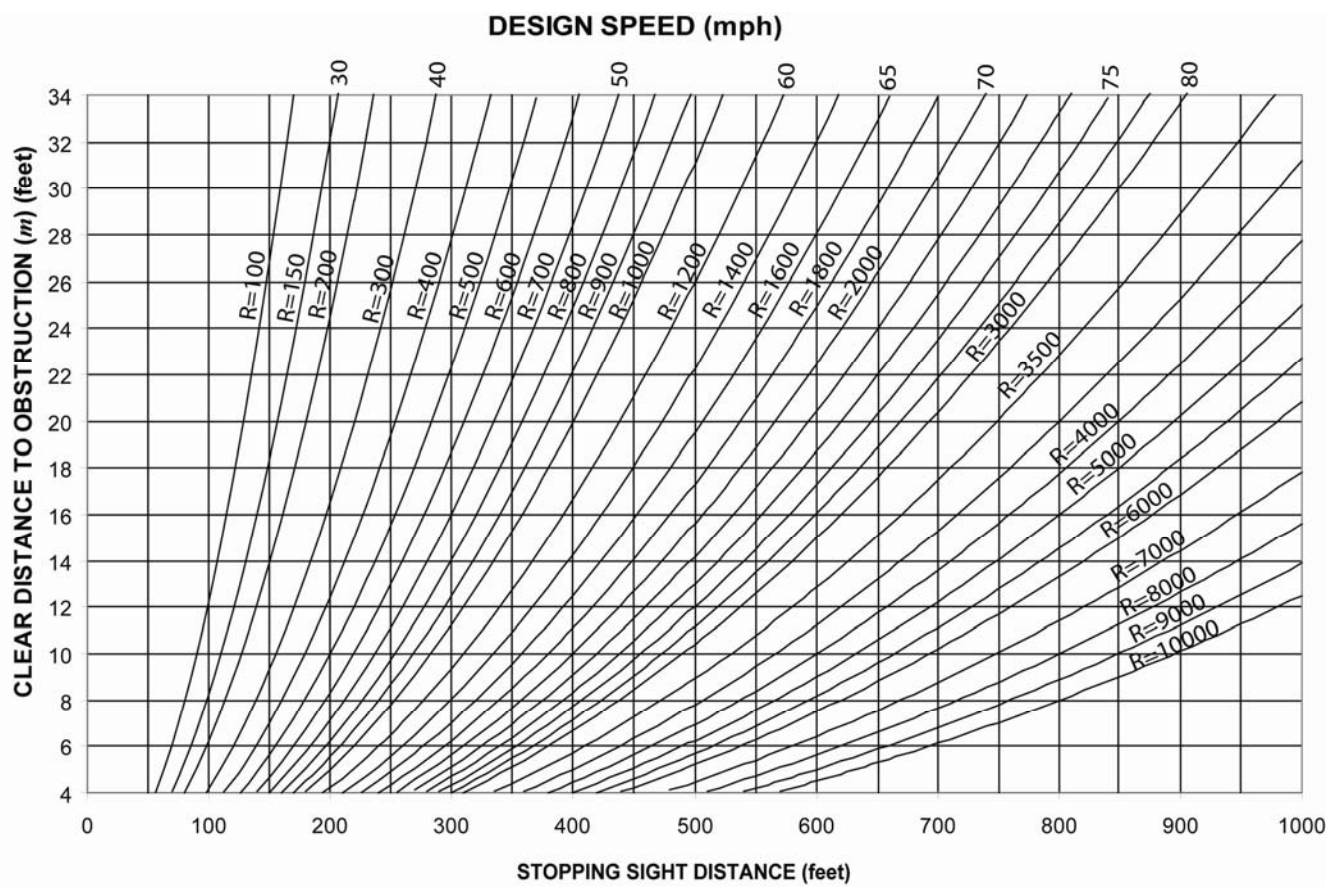
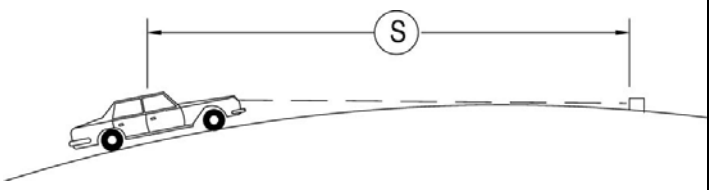


Figure 201.7
Decision Sight Distance on Crest Vertical Curves



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

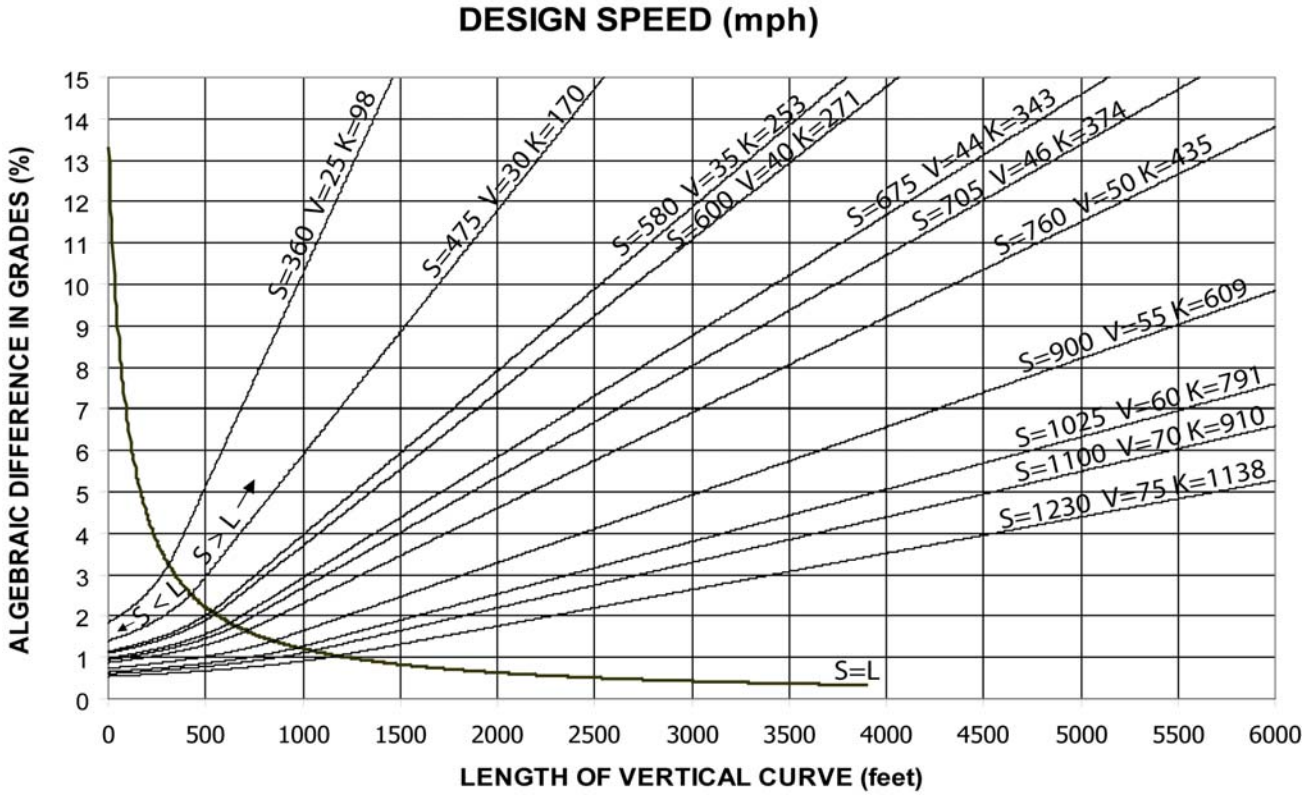
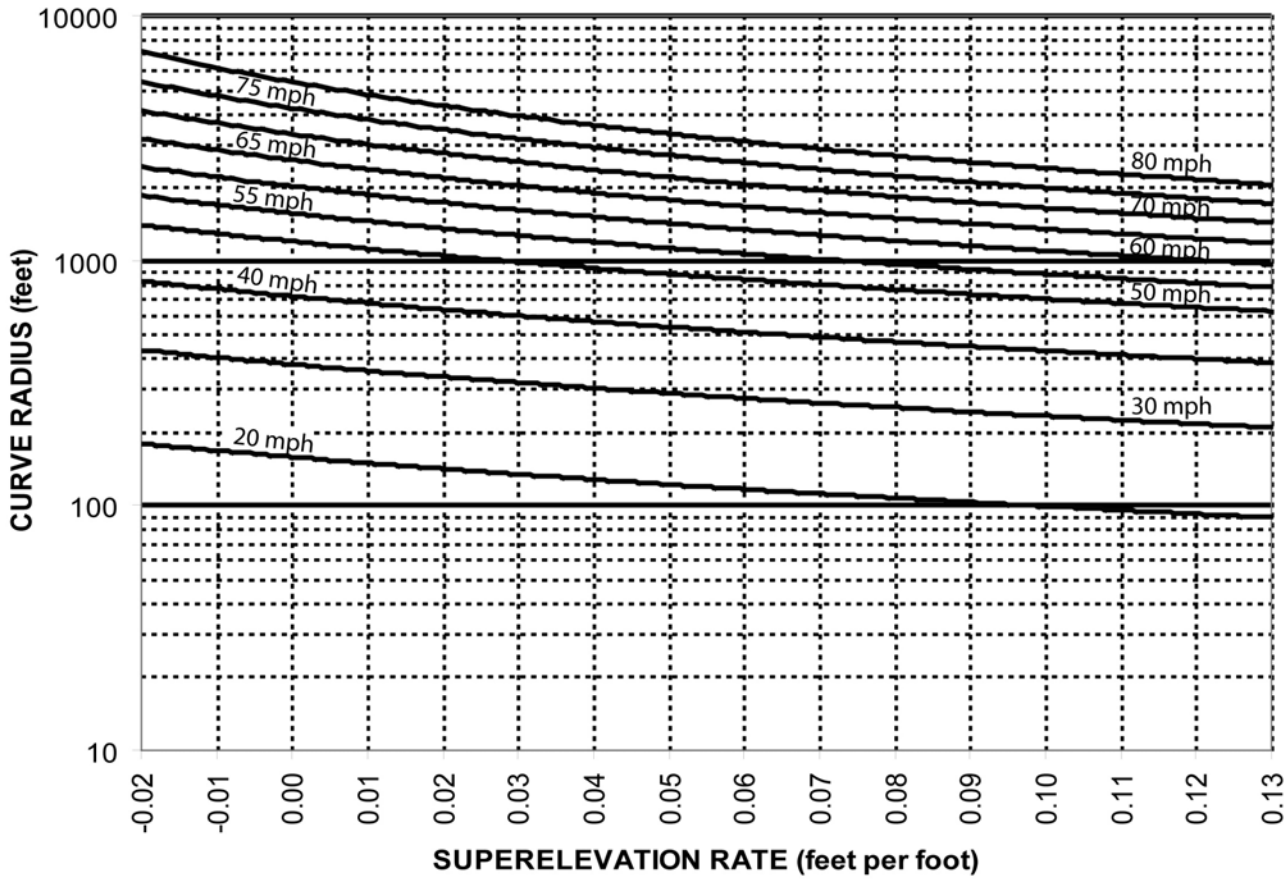


Figure 202.2
Maximum Comfortable Speed on Horizontal Curves



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)

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

(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.

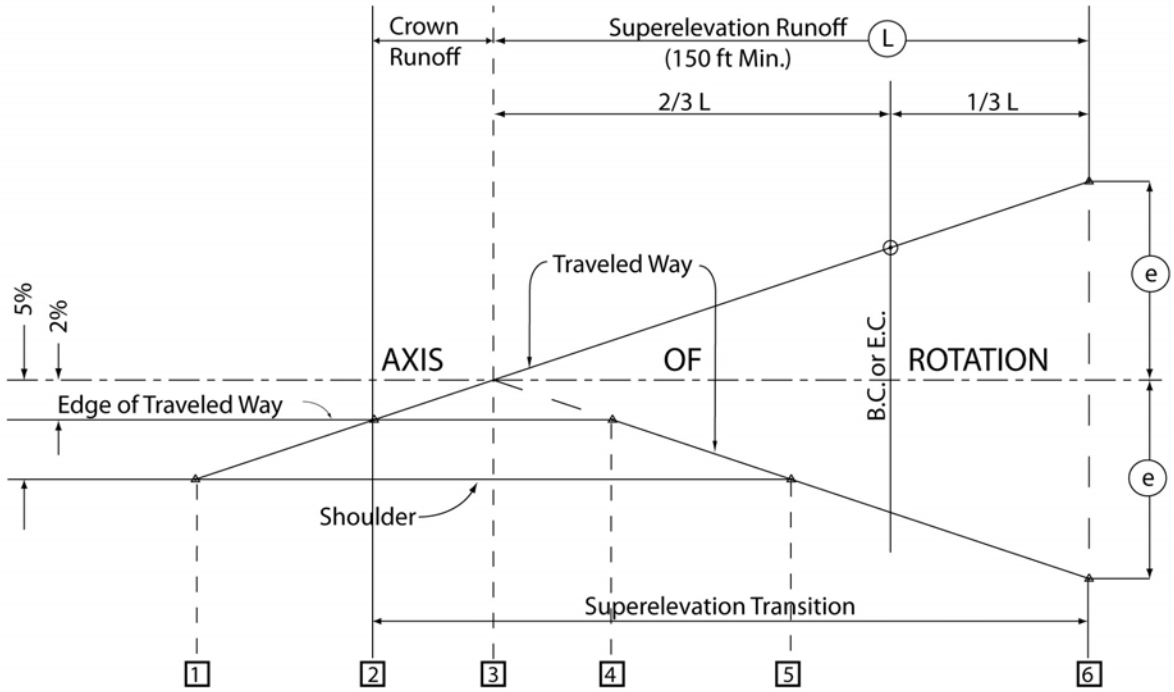
203.2 Standards for Curvature

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

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



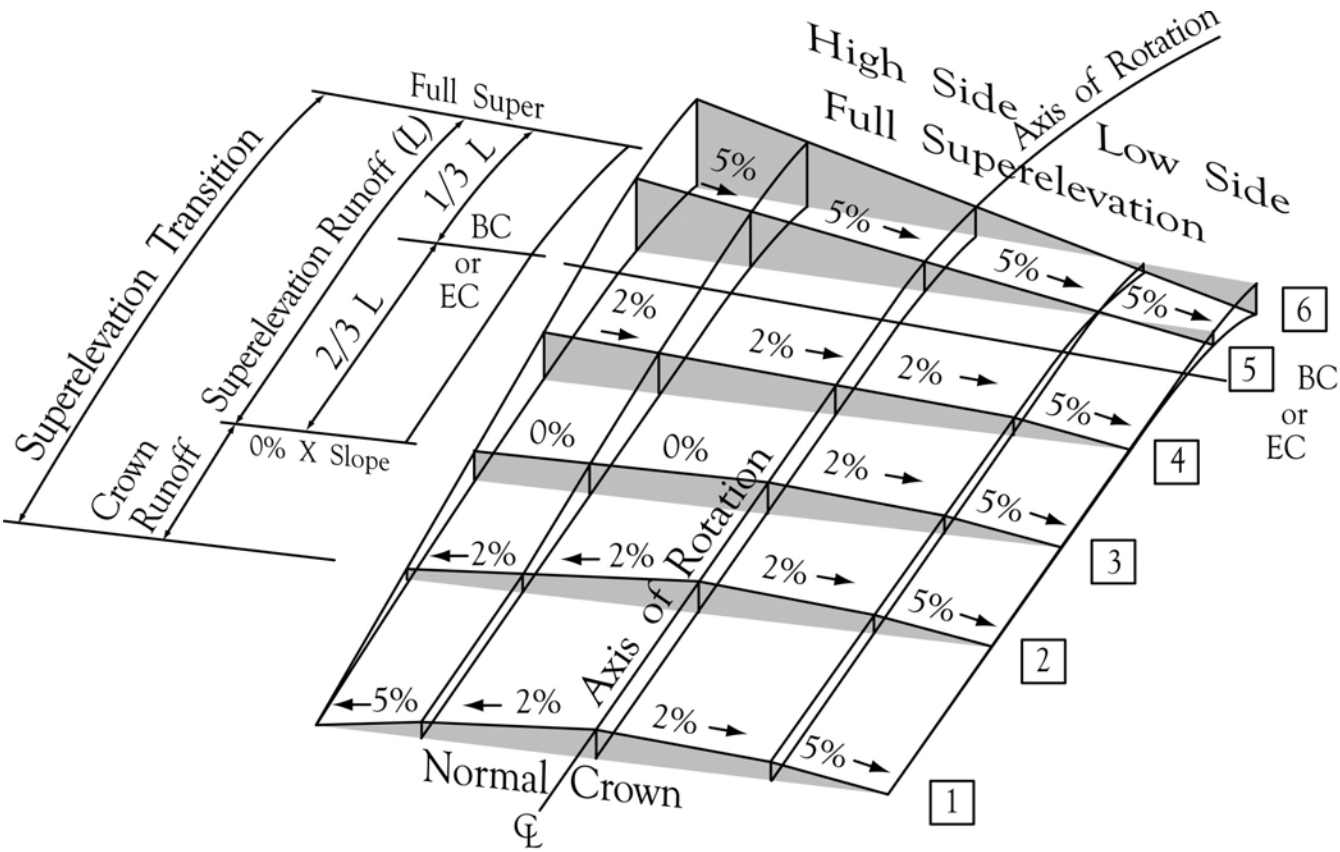
Superelevation Runoff Lengths

Superelevation Rate "e" ft/ft	Length, L (feet)								
	2-Lane Highways & Multilane Ramps	Single Lane Ramps	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.
% On tangent	The percentage of the superelevation runoff length (L) that is outside of the curve (2/3L). See Index 202.5(2).
% 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)

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.* 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. When several access openings are closely spaced, a frontage road should be considered (see Index 104.3). To discourage wrong-way movements, access openings should be located directly opposite, or at least 300 feet from a median opening.

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).

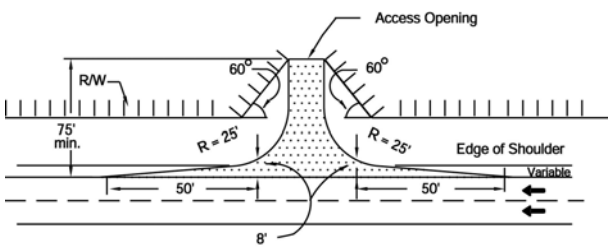
(3) *Recessed Access Openings.* Recessed access openings, as shown on Figure 205.1, are desirable at all points where private access is permitted and should be provided whenever they can be obtained without requiring alterations to existing adjacent improvements. When recessed openings are required, 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 and Disabled Persons Access. Where sidewalks traverse driveways, accessibility regulations require that a relatively level (2 percent max. cross fall) path, at least 4 feet wide, is provided. 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:

- Where restricted parking zones have been established (either blue or white painted zones) adjacent to driveways, but no reasonably close ramp access to the sidewalk exists, consideration should be given to reducing the maximum slope of the driveway from 10 percent to 8.33 percent to provide sidewalk access to the disabled.
- 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 pedestrian activity is neither present, nor expected to be present within the reasonable future, 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).

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 drivers expectations. Transition tapers for these types of situations should be at 10:1 (longitudinal to lateral).

(4) *Shoulder Widening.* Shoulder widening should normally be accomplished in a manner that provides a smooth transition, but can be accomplished without a taper if necessary.

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 504.2L, 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 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.

CHAPTER 300
GEOMETRIC CROSS SECTION

Topic 301 - Traveled Way Standards

Index 301.1 - Traveled Way Width

The traveled way width is determined by the number of lanes demanded by the design hourly volume. The traveled way width does not include curbs, dikes, gutters, or gutter pans. **The basic lane width for new construction on two-lane and multilane highways, ramps, collector roads, and other appurtenant roadways shall be 12 feet.** For roads with curve radii of 300 feet or less, widening due to offtracking should be considered. See Index 404.1 and Table 504.3A. For roads under other jurisdictions, see Topic 308.

301.2 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.

(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 when necessary to match existing cross slopes, the minimum shall be 1.5 percent and the maximum shall be 3 percent.** However, the cross slope on 2-lane and multilane AC highways should be increased to 2 percent if the cost is reasonable.

(c) **On unpaved roadway surfaces, including gravel and penetration treated earth, the cross slope shall be 2.5 percent to 5.0 percent.**

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

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

On divided highway roadbeds, the high point of crown may be centered at, or left of, the center of the traveled way, and preferably over a lane line (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 - Shoulder Standards

302.1 Width

The shoulder widths given in Table 302.1 shall be the minimum continuous usable width of paved shoulder. For new construction, and major reconstruction projects on conventional highways, adequate width should be provided to permit shared use by motorists and bicyclists.

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.

Table 302.1
Standards for Paved Shoulder Width

	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
6-lanes or more	8	8
Urban areas with 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. 4 feet preferred in urban areas and/or when ramp is metered. See Index 504.3.
(3) In restrictive situations, may be reduced to 2 feet or 4 feet (preferred in urban areas) in the 2-lane section of a non-metered ramp which transitions from a single lane. May be reduced to 2 feet in ramp sections having 3 or more lanes. See Index 504.3.
(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 1003.2 if bike lanes are present.
(6) 10-foot shoulders preferred.
(7) Where parking is allowed, 10 feet to 12 feet shoulders preferred.
(8) Shoulders adjacent to abutment walls, retaining walls in cut locations, and noise barriers shall be 10 feet wide.

Table 303.1
Selection of Curb Type

Location	Operating Speeds (mph)		
	≤ 40	45	≥ 50
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(2)	A or B-6	B-6	B-4 or D
- Adjacent to Sidewalks & Pedestrian Refuge Islands	A (3)	A-6	B-6
- Bulb outs/curb extensions	A (3)	NA	NA
- Bridges (4)	H, A3, or B3	H or B3	B3

(1) Based on the operating speed along the frontage road.
(2) See Design Information Bulletin Number 80, "Roundabouts" 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.

The curb sections provided on the Standard Plans are approved types to be used as stated below. The following types are vertical curb:

(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 (operating speed ≤ 40 miles per hour). These curbs do not constitute a positive barrier as they can be 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 operating speeds are 40 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 with operating speeds less than or equal to 45 miles per hour 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 and Curbs with Gutter Pan.* 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 operating speeds equal or exceed 50 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.

Where bicycles are permitted and the shoulder width is 4 feet, 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. For mandatory requirements regarding drainage inlet grates for bicycles, see Index 1003.6(3).

303.3 Dike Types and Uses

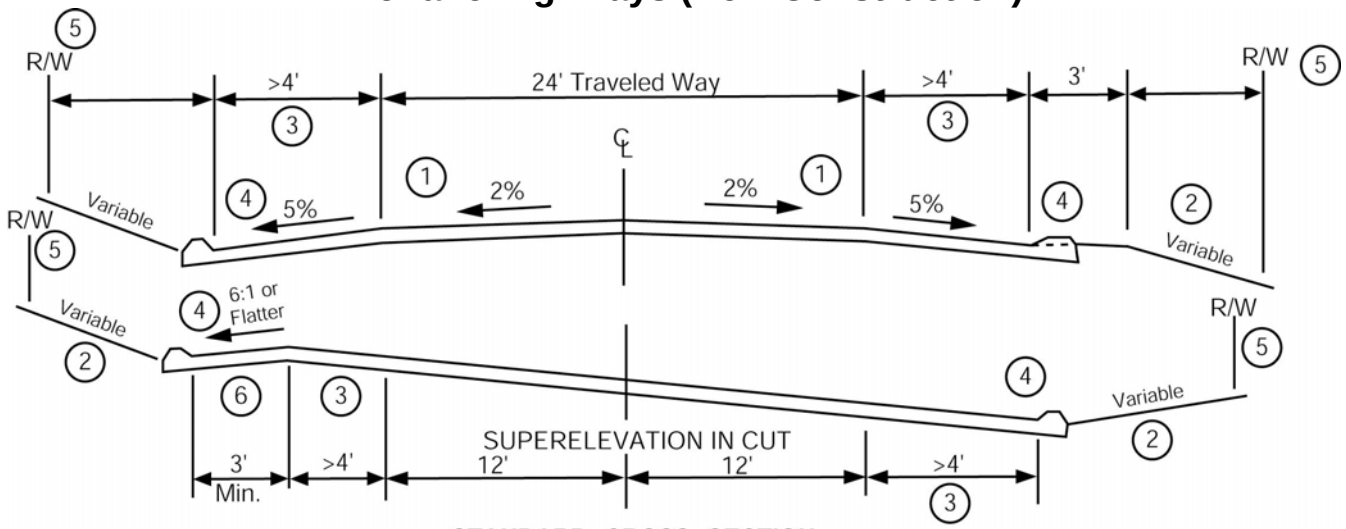
Use of dike is intended for drainage control and should not be used in place of curb. Dikes placed adjoining the shoulder, as shown in Figures 307.2, 307.4, and 307.5, provide a paved triangular gutter within the shoulder area. The dike sections provided on the Standard Plans are approved types to be used as stated below. Dikes should be selected as illustrated in Figure 303.3. 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.

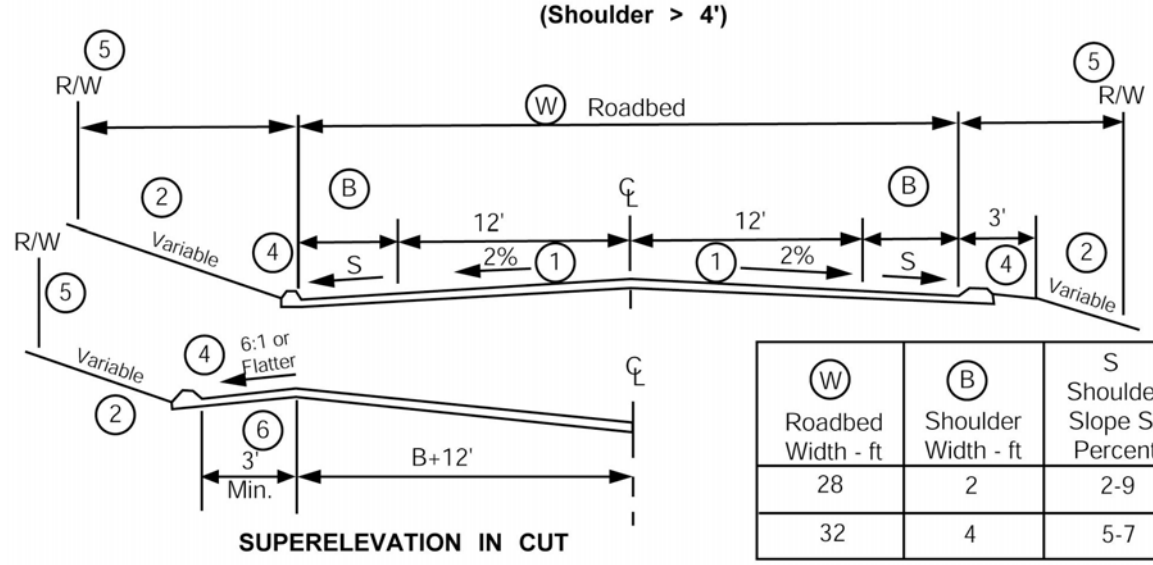
(2) *Type C Dike.* This low dike, 2 inches in height, may be used to confine small concentrations of runoff. The capacity of the shoulder gutter formed by this dike is small. Due to this limited capacity, the need for installing an inlet immediately upstream of the beginning of this dike type should 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 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 is more 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.

Figure 307.2
Geometric Cross Sections for
Two-lane Highways (New Construction)



STANDARD CROSS SECTION
(Shoulder > 4')



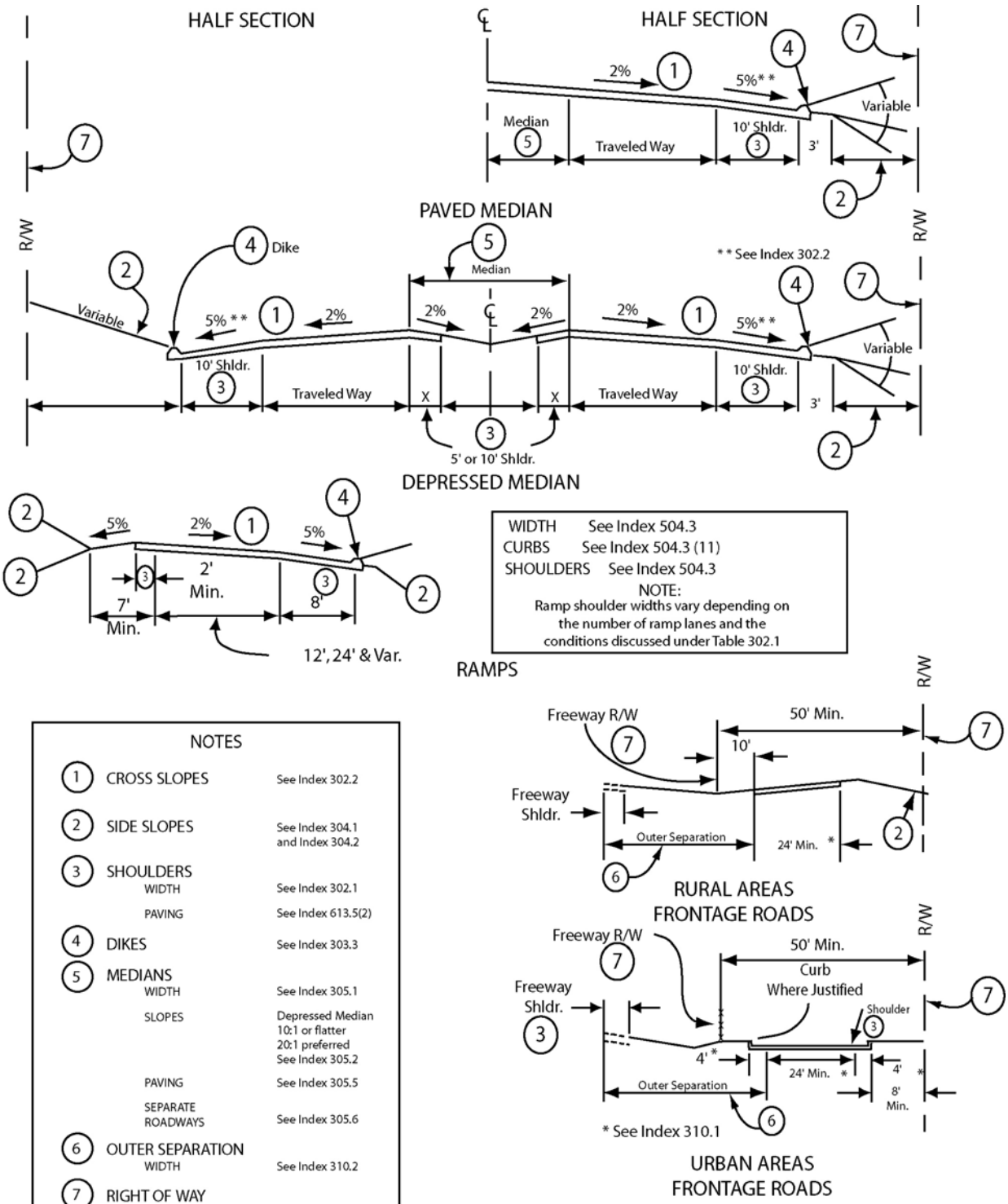
W Roadbed Width - ft	B Shoulder Width - ft	S Shoulder Slope S-Percent
28	2	2-9
32	4	5-7

SUPERELEVATION IN CUT

ALL PAVED 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



NOTES

1	CROSS SLOPES	See Index 302.2
2	SIDE SLOPES	See Index 304.1 and Index 304.2
3	SHOULDERS	
	WIDTH	See Index 302.1
	PAVING	See Index 613.5(2)
4	DIKES	See Index 303.3
5	MEDIANS	
	WIDTH	See Index 305.1
	SLOPES	Depressed Median 10:1 or flatter 20:1 preferred See Index 305.2
	PAVING	See Index 305.5
	SEPARATE ROADWAYS	See Index 305.6
6	OUTER SEPARATION	
	WIDTH	See Index 310.2
7	RIGHT OF WAY	
	WIDTH	See Index 306.1
8	FRONTAGE ROADS	See Index 310.1

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 40 mph and curbs, clear recovery zone widths do not apply. See minimum horizontal clearance, Index 309.1(3)(c).

Fixed objects including 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:

(a) Fixed objects should be eliminated or moved outside the clear recovery zone to a location where they are unlikely to be hit.

(b) If sign posts six inches or more in any dimension or light standards cannot be eliminated or moved outside the clear recovery zone, they should be made yielding with a breakaway feature.

(c) If a fixed object cannot be eliminated, moved outside the clear recovery zone, or modified to be made yielding, it should be shielded by guardrail or a crash cushion.

Shielding 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.

Where compliance with the above stated clear recovery zone guidelines are impractical, the minimum horizontal clearance cited below 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 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 roads, shall not be less than 10 feet.**

(c) **On conventional highways, frontage roads, city streets and county roads (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 feet 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 District Permit Engineer.

The Regional Permit Manager should 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.

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.).**

(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 will be subjected to extensive review by FHWA and must be approved by the Military Traffic Management Command Traffic Engineering Agency (MTMCTEA) in Washington D. C. 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/MTMCTE notification. See Robert L. Buckley's memo dated March 30, 2000 to District Directors for more information on this subset of the Interstate system.

(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

CHAPTER 400
INTERSECTIONS AT GRADE

Topic 401 - Factors Affecting Design

Index 401.1 - General

At-grade intersections must handle a variety of conflicts among vehicles, pedestrians, and bicycles. These recurring conflicts, a unique characteristic of intersections, play a major role in the preparation of design standards and guidelines. Arriving, departing, merging, turning, and crossing paths of moving traffic have to be accommodated within a relatively small area.

401.2 The Driver

The assumption of certain driver skills is a factor in intersection design. A driver's perception and reaction time set the standards for sight distance and length of transitions.

401.3 The Vehicle

Size and maneuverability of vehicles are factors that influence the design of an intersection.

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.

401.4 The Environment

In highly developed urban areas, street parking, pedestrians, and transit buses add to the complexity of a busy intersection.

Industrial development may require special attention to the movement of large trucks.

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

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.

401.5 The Pedestrian

Pedestrian considerations are an integral part of intersection design because of their potential conflict with motor vehicles. Such factors include expected pedestrian volumes, age ranges, and physical abilities, etc. Geometric features which may affect the pedestrian should be taken into account. See Topic 105 Pedestrian Facilities, and the California Manual on Uniform Traffic Control Devices (California MUTCD).

401.6 The Bicyclist

The presence of bicyclists on State routes should be considered early in design. Chapter 1000 gives information on bikeway planning and design criteria.

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.* Chapter 10 of the "Highway Capacity Manual", gives 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. See the California MUTCD, Chapter 4C for signal warrants.

(2) *Signalized Intersections.* See Topic 406 for analysis of simple signalized intersections, including ramps. The analysis of complex signalized intersections should be referred to the District Traffic Branch.

402.2 Accidents

(1) *General.* Intersections have a higher potential for conflicts compared to other sections of the highway. At an intersection continuity of travel is interrupted, traffic streams cross, and many types of turning movements occur.

The type of traffic control affects the type of accidents. Signalized intersections tend to have more rear enders and same-direction sideswipes than stop-controlled intersections. The latter tend to have more angle or crossing accidents due to a lack of positive control.

(2) *Undesirable Geometric Features.*

- Inadequate approach sight distance.
- Inadequate corner sight distance.
- Steep grades.
- Inappropriate traffic control.
- Five or more approaches.
- Presence of curves within intersections.

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 intersectional areas are usually undesirable. The hazards of conflicting movements are magnified when drivers and bicyclists are unable to anticipate movements of other traffic 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.

Large areas of intersectional conflicts are characteristic of skewed intersection angles. Therefore, angles of intersection approaching 90° will aid in reducing conflict areas.

403.3 Angle of Intersection

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

- The shortest crossing distance for motor vehicles, bicycles, and pedestrians.
- Sight lines which optimize corner sight distance and the abilitiy of drivers to judge the relative position and speed of approach vehicles.

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

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 waiting at the crossroad and the driver of an approaching vehicle.

Adequate time must be provided for the waiting vehicle 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 on the crossroad shall be a minimum of 10 feet plus the shoulder width of the major road but not less than 15 feet.** Corner sight distance is to be measured from a 3.5-foot height at the location of the driver on the minor road to a 4.25-foot object height in the center of the approaching lane of the major road. 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)--<u>At unsignalized public road intersections (see Index 405.7) corner sight distance values given in Table 405.1A should be provided.</u>

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 vehicle conflicts can occur due to violation of signal, right turns on red, malfunction of the signal, or use of flashing red/yellow mode.

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.

(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
- 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 and the Design Coordinator.

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

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

(1) Using stopping sight distance between an eye height of 3.5 ft and an object height of 4.25 ft. See Index 405.1(2)(a) for setback requirements.
(2) Apply corner sight distance requirements at signalized intersections whenever possible due to unanticipated violations of the signals or malfunctions of the signals. See Index 405.1(2)(b).

405.2 Left-turn Channelization

(1) General. The purpose of a left-turn lane is to expedite the movement of through traffic, control the movement of turning traffic, increase the capacity of the intersection, and improve safety characteristics.

The District Traffic Branch normally establishes the need for left-turn lanes. See "Guidelines for Reconstruction of Intersections," August 1985, published by the California Division of Transportation Operations.

(2) Design Elements.

(a) Lane Width -- **The lane width for both single and double left-turn lanes on State highways shall be 12 feet.** Under certain circumstances (listed below), left-turn lane widths of 11 feet or as narrow as 10 feet may be used on RRR or other projects on existing State highways and on roads or streets under other jurisdictions when supported by an approved design exception pursuant to Index 82.2. 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.

- On high speed rural highways or moderate speed suburban highways where width is restricted, the minimum width of single or dual left-turn lanes may be reduced to 11 feet.
- In severely constrained situations on low to moderate speed urban highways where large trucks are not expected, the minimum width of single left-turn lanes may be reduced to 10 feet. When double left-turn lanes are warranted under these same circumstances the width of each lane shall be no less than 11 feet. This added width is needed to assure adequate clearance between turning vehicles.

(b) Approach Taper -- On a conventional highway 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.

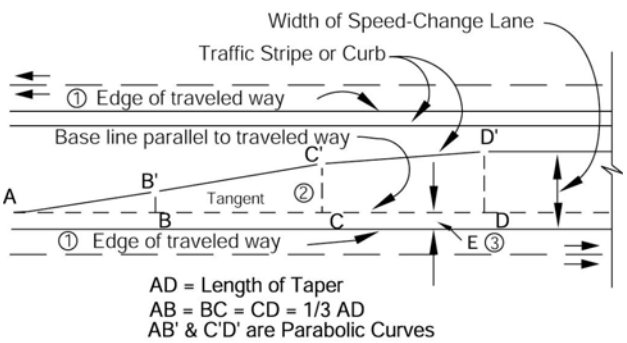
Figures 405.2B and 405.2C show alternate designs foreshortened with the deceleration lane beginning at the 2/3 point of the approach taper so that part of the deceleration takes place in the through traffic lane. Figure 405.2C is shortened further by widening half (or other appropriate fraction) on each 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 mph to 20 mph 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.

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

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

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. As a minimum, storage length should be calculated 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.

(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

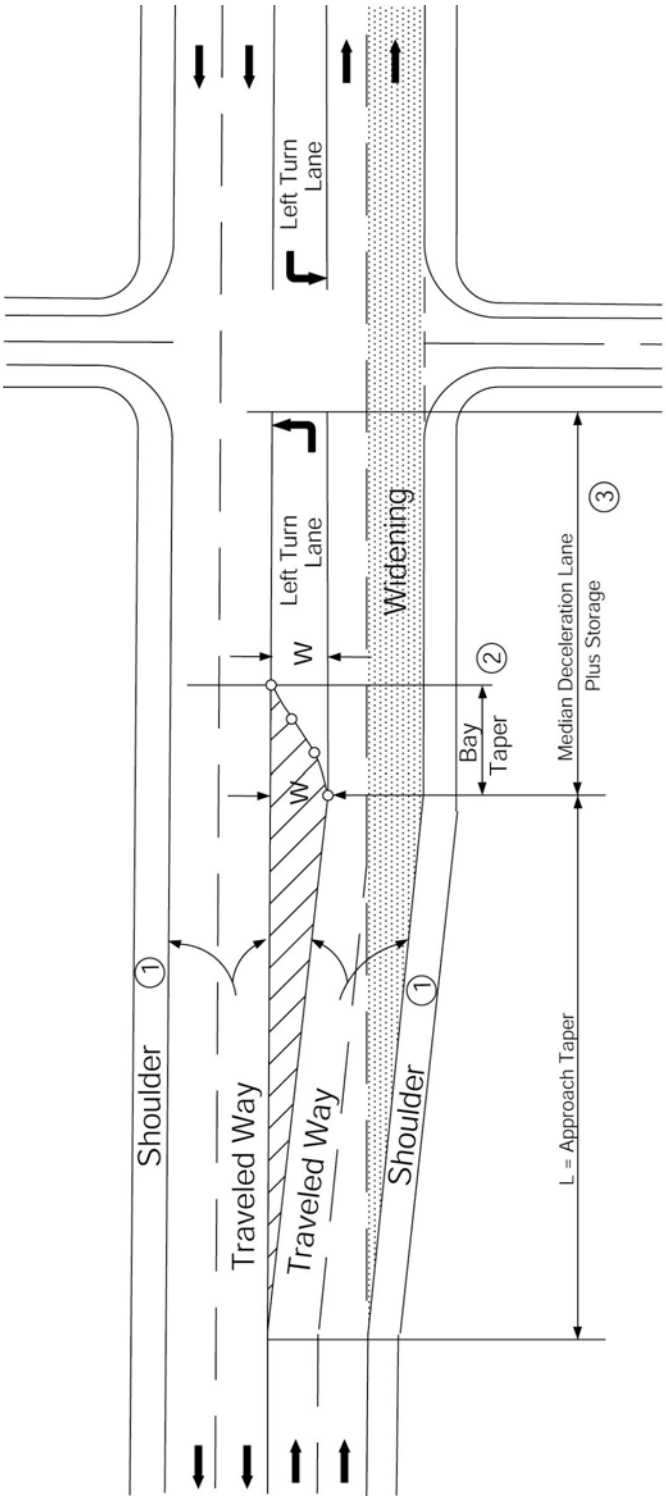
(1) *General.* For right-turning traffic, delays are less critical and conflicts less severe than for left-turning traffic. Nevertheless, right-turn lanes can be justified on the basis of capacity, analysis, and accident experience.

In rural areas a history of high speed rear-end accidents 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.
- Pedestrians conflicting with right turning vehicles.
- Frequent rear-end and sideswipe accidents involving right-turning vehicles.

Figure 405.2A
Standard Left-turn Channelization



EQUATION: L = Use WV, for V≥45mph ④
Or $WV^2/60$, for V<45mph

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.

(2) Design Elements.

(a) Lane and Shoulder Width--**The basic lane width for right-turn lanes shall be 12 feet. Shoulder width shall be a minimum of 4 feet.** Whenever possible, consideration should be given to increasing the shoulder width to 8 feet to facilitate the passage of bicycle traffic and provide space for vehicle breakdowns. Although not desirable, lane and shoulder widths less than those given above can be considered for right-turn lanes under the following conditions and with the approval of a design exception pursuant to Index 82.2.

- On high speed rural highways or moderate speed suburban highways where width is restricted, consideration may be given to reducing the lane width to 11 feet with approval of a design exception.
- On low to moderate speed roadways in severely constrained situations, consideration may be given to reducing the minimum lane width to 10 feet with approval of a design exception.
- Shoulder widths may also be considered for reduction under constricted situations. Whenever possible, at least a 2-foot offset should be provided where the right-turn lane is adjacent to a curb. Entire omission of the shoulder should only be considered in the most severely constricted 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 lane width.

Additional right of way for a future right-turn lane should be considered when an intersection is being designed.

(b) 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.

(c) 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 mph to 20 mph for a lower entry speed. For example, if the main line speed is 50 mph and a 10 mph deceleration is permitted on the through lanes, the deceleration length may be that required for 40 mph.

(d) 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 cause problems 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. Also, rear-end accidents 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.

Free right turns should generally be avoided unless there is room for a generous acceleration lane or a lane addition on the local street. See Index 504.3(3) for additional information.

405.4 Traffic Islands

A traffic island is an area between traffic lanes for control of vehicle movements or for pedestrian refuge. An island may be designated by paint, raised pavement markers, curbs, pavement edge, or other devices. Examples of traffic island designs are shown on Figure 405.4.

Traffic islands usually serve more than one function, but may be generally classified in three separate types:

(a) Channelizing islands which are designed to confine specific traffic movements into definite channels;

(b) Divisional islands which serve to separate traffic moving in the same or opposite direction; and

(c) Refuge islands to aid and protect pedestrians crossing the roadway. If a divisional island is located in an urban area where pedestrians are present, portions of each island can be considered a refuge island.

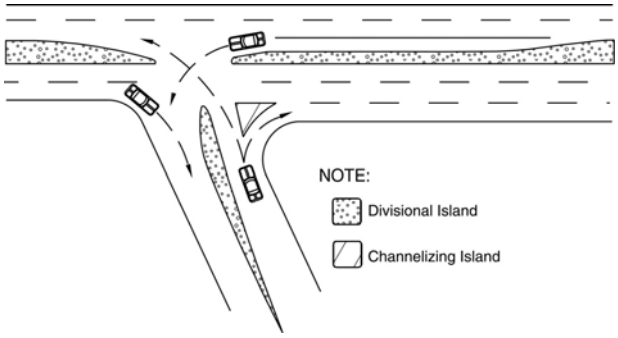
Traffic islands are also used to discourage or prohibit undesirable movements.

(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 islands should not be less than 4 feet wide and 20 feet long.

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. These offsets are in addition to the normal 2-foot left and 8-foot right shoulder widths. 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.

Figure 405.4
Traffic Island Designs



(2) Delineation of Traffic Islands. Generally, islands should present the least potential conflict to approaching vehicles and yet perform their intended function. When curbs are used, Type B is preferable except where a Type A curb is needed for traffic control or pedestrian refuge (see Index 303.2). Islands may be designated as follows:

(a) Raised paved areas outlined by curbs.

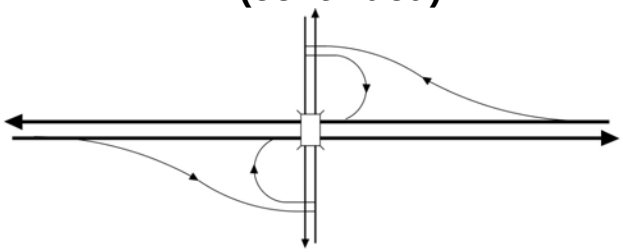
(b) Flush paved areas outlined by pavement markings.

(c) Unpaved areas (small unpaved areas should be avoided).

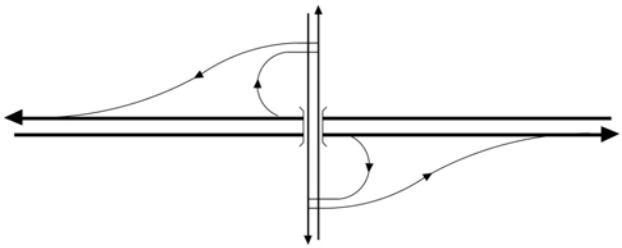
On facilities with speeds over 45 mph, the use of any type of curb is discouraged. Where curbs are to be used, they should be located at or outside of the shoulder edge, as discussed in Index 303.5.

In rural areas, painted channelization supplemented with raised pavement markers would be more appropriate than a raised curbed channelization. The design is as forgiving as possible and decreases the consequence of a driver's failure to detect or recognize the curbed island.

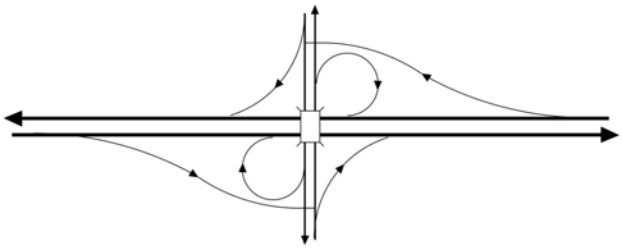
Figure 502.2
Typical Local Street Interchanges
(continued)



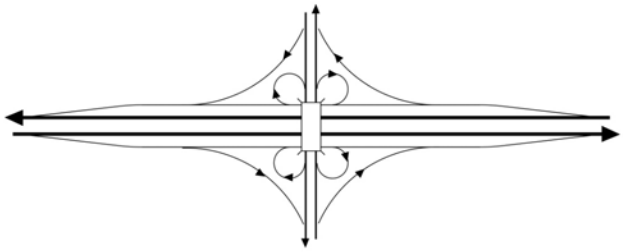
TYPE L-7



TYPE L-8

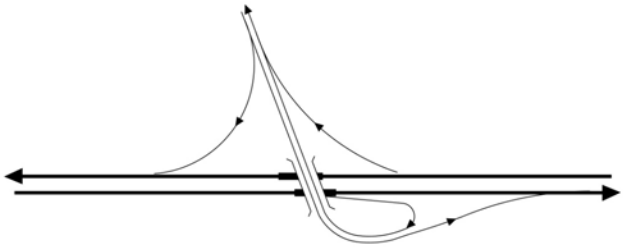


TYPE L-9

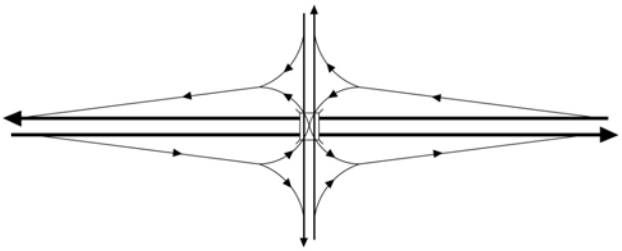


TYPE L-10

TYPE L-11



TYPE L-12



TYPE L-13

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

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, deceleration prior to the curve or end of anticipated queue, and adequate superelevation for anticipated driving speeds can be developed.

(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 volume of interchange traffic which can be accommodated on the crossroads.

The four-quadrant cloverleaf interchange (Type L-10) has free-flow characteristics for all movements. It has the disadvantage of a higher cost than a diamond or partial cloverleaf design and a relatively short weaving section between the loop ramps which limits capacity. 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. Detailed information on SPI's is provided in the Single Point Interchange Planning, Design and Operational Guidelines (SPI Guidelines), originally issued by memorandum on June 15, 2001. Per the SPI Guidelines, the Design Coordinator and the Headquarters Traffic Liaison must approve the SPI concept.

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.

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 optimum highway system. Parameters such as cost, environment, community values, traffic volumes, route continuity, map relatability, and safety should all be considered. Both the sign route and the major traffic volume should be to the left at a freeway-to-freeway interchange, if possible.

(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) Elimination of Connections--Freeway-to-freeway interchanges need not include all possible turning movements. Connections serving minor traffic volumes or significantly out-of-direction traffic movements should be omitted unless it can be demonstrated that traffic service and other benefits justify the costs. Considerations include:

- Traffic volumes--Turning traffic volumes may be nominal or a small percentage of the total interchanging traffic.
- Circuitous--Connections may only serve significantly out-of-direction traffic movements.
- Freeway location--Where three freeways cross so as to form a relatively small triangle, the omission of the backward freeway-to-freeway

connections from one leg of the triangle to another may have little negative impact on local or through traffic service.

- Use of local streets--Low turning volumes may be accommodated reasonably well by way of local interchanges and the local street system. There may be both traffic operational advantages and economic savings from utilizing and improving this local system in lieu of providing the freeway-to-freeway connections.
- Staging--Staging possibilities should be thoroughly assessed. Provisions should be made for adding or upgrading ramps and connectors at a later time. For example, an initial loop ramp might be later upgraded to direct connector.
- Effect on other traffic movements--Provision of minor movements may be detrimental to traffic operation on major branch connections and the main line freeways.
- Costs--All construction and right of way savings and costs attributable to the elimination of turning movements should be considered. This includes possible additional local interchange and street costs as well as reductions in the freeway-to-freeway interchange costs.
- Signing--Freeway-to-freeway traffic may be signed via the local street system. Routes should be sufficiently direct and well oriented to insure that the unfamiliar driver can follow them.

(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.

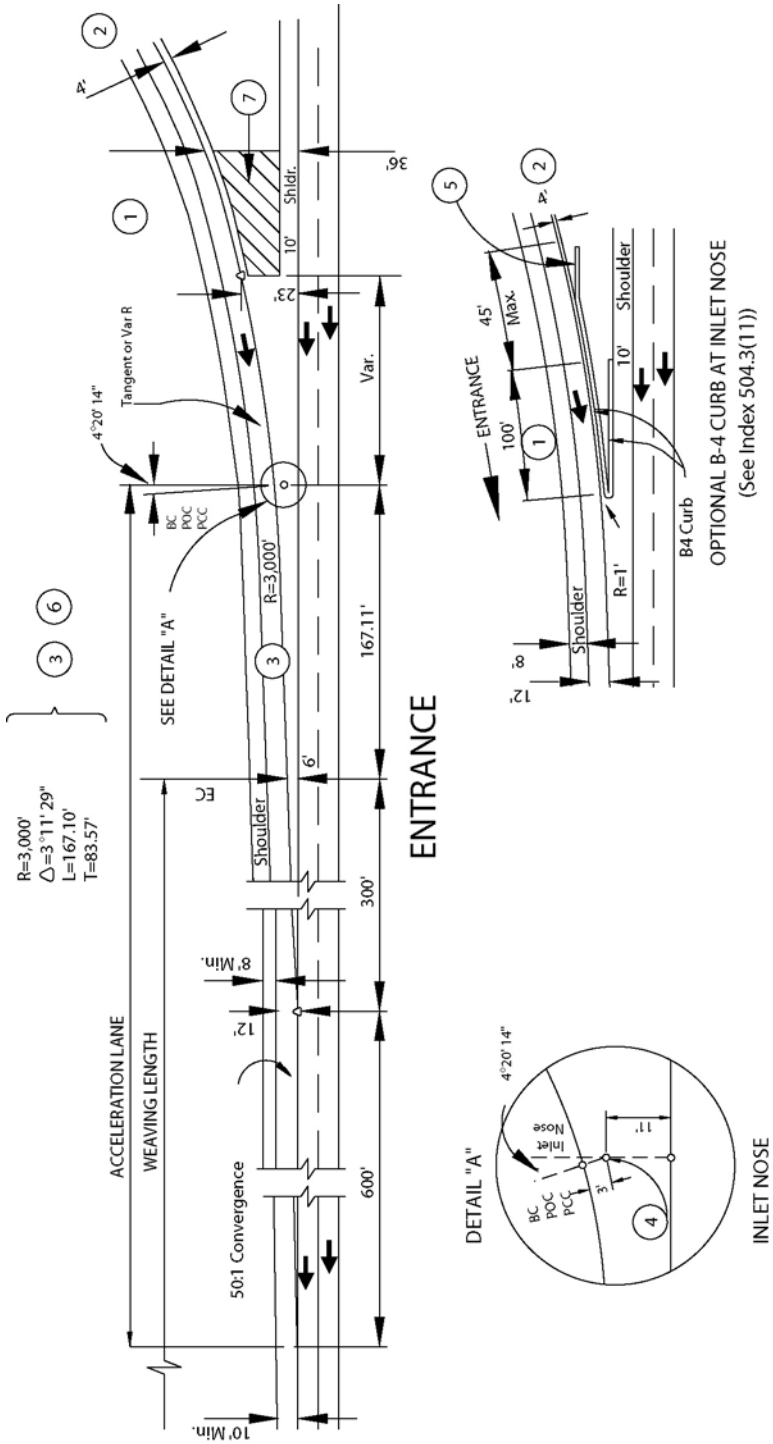
(e) Alignment--It is not considered practical to establish fixed freeway-to-freeway interchange alignment standards. An interchange must be designed to fit into its environment. Alignment is often controlled by external factors such as terrain, buildings, street patterns, route adoptions, and community 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

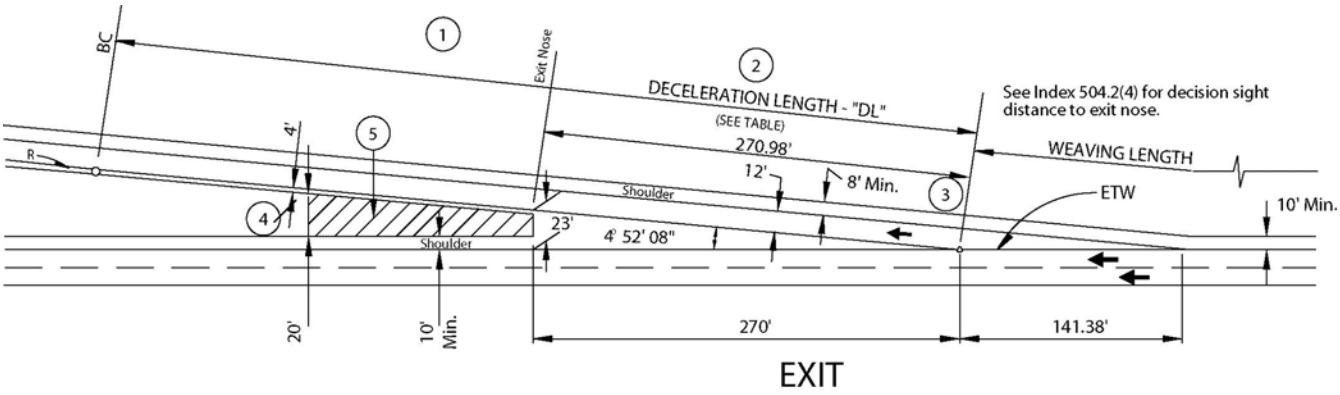
Figure 504.2A
Single Lane Freeway Entrance



NOTES:

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

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)

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).

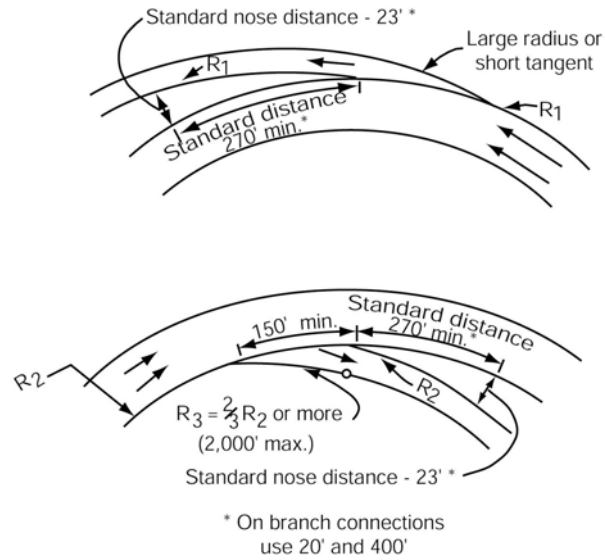
On entrance ramps the distance from the inlet nose (14-foot point) to the end of the acceleration lane taper should equal the sum of the distances shown on Figure 504.2A. The 50:1 (longitudinal to lateral) taper may be curved to fit the conditions, and the 3,000-foot radius curve may be adjusted (see 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 insure 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 representatives of the Headquarters Division of Traffic Operations and the Division of Design. Also, see Index 204.5 "Sustained Grades".

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 ETW**

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 504.3A 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 bus or truck usage of that lane.

Table 504.3A
Ramp Widening for Trucks

Ramp Radius (ft)	Widening (ft)	Lane Width (ft)
<150	6	17
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). Where conditions preclude the use of a 50 to 1 taper, the lane should be dropped using a taper of no less than 30 to 1. **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 Design Coordinator or Design Reviewer from the Division of Design. **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 new facilities should normally be based upon the projected peak-hour traffic volumes 20 years after completion of construction, except as stated in Index 103.2.

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 multi-lane ramp segment may be provided to increase vehicle storage within the available ramp length (see 504.3(2)(d) Storage Length) and/or to create a preferential lane for HOVs, as required in Section 504.3(2)(h).

(b) Metered Multi-Lane Entrance Ramps

When entrance ramp volumes exceed 900 vph, and/or when an HOV lane is determined to be necessary, 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 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 the 300 feet.

The recommended widths for metered ramps are shown in Table 504.3B.

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.

Table 504.3B
Pavement Widths

Metered Ramp	Traveled Way	Inside Shoulder	Outside Shoulder
1-lane	12 ft	4 ft	8 ft
2-lane	24 ft	4 ft	8 ft
3-lane	36 ft	2 ft	2 ft

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 geometrics and speed, the lane drop transition should be accomplished with a taper of between 30 and 50:1. **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

related to the design of HOV preferential lane access.

Signing for an 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 extinguishable 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 trapping Single-Occupancy Vehicles (SOVs) in an 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 an 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 examples: better control over ramp traffic can be attained by retrofitting ramps to meter HOV traffic which bypasses the ramp meter (District 3, 7, and 12). 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 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 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 vehicle occupancy requirements. At locations where the HOV lane is metered, the 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 enforcement area should be downstream of the mast arm standard, approximately 70 feet from the limit line. The length of the 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 shall coordinate enforcement issues with the California Highway Patrol. The CHP Area Commander shall be contacted during the Project Report stage, prior to design, to discuss any variations needed to the enforcement area designs shown in this manual. Variations shall be discussed with the HQ Traffic Liaison and the Design Coordinator and/or Design Reviewer.

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, circuitous of 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 intersections.

Ramp terminals should connect where the grade of the overcrossing is 4 percent or less to avoid potential overturning of trucks.

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.

Where a separate right-turn lane is provided at ramp terminals, the turn lane should not continue as a "free" right unless pedestrian volumes are low, the right-turn lane continues as a separate full width lane for at least 200 feet prior to merging and access control is maintained for at least 200 feet past the ramp intersection. Provision of the "free" right should also be precluded if left-turn movements of any kind are allowed within 400 feet of the ramp intersection.

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).

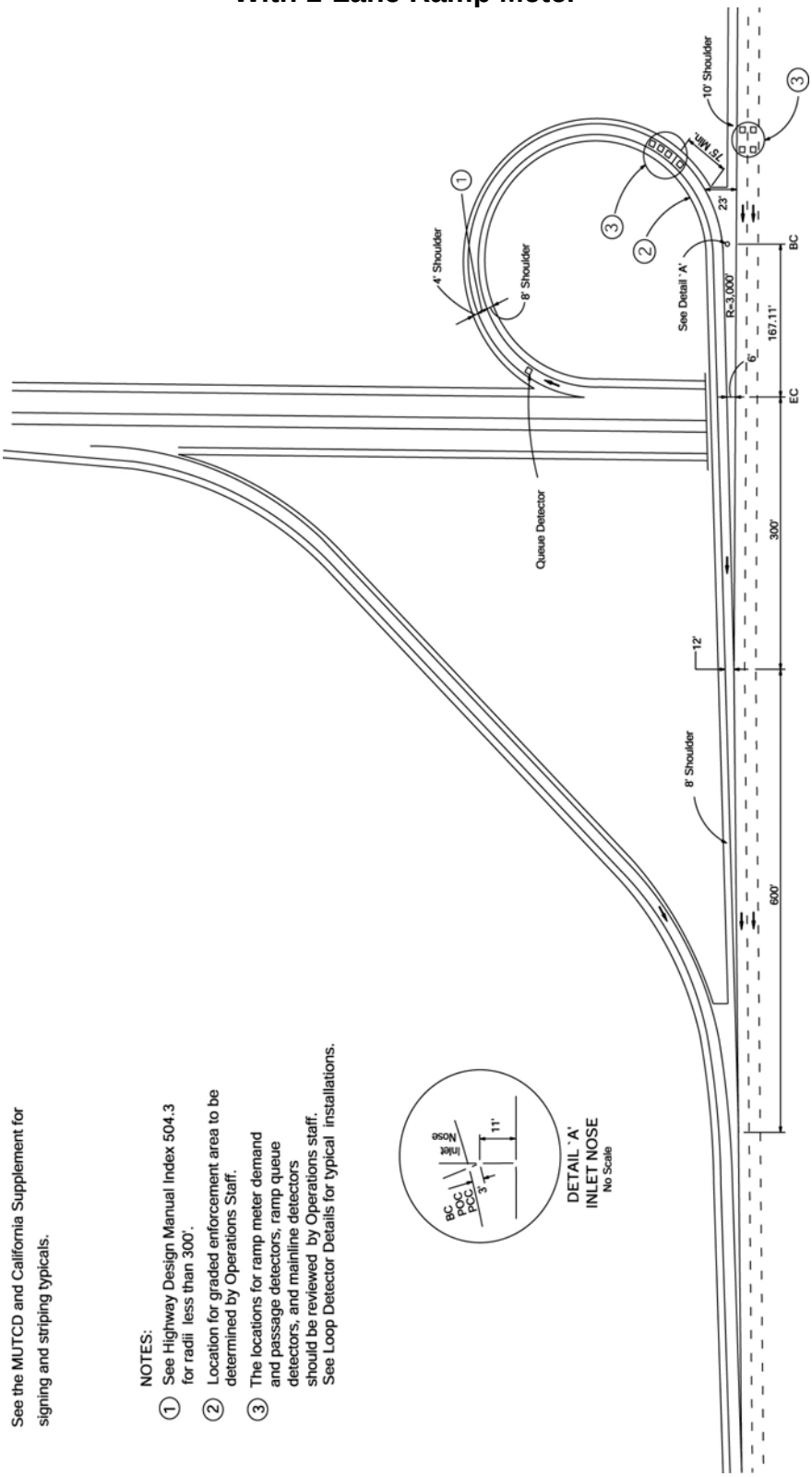
For additional information on sight distance requirements at signalized intersections, see Index 405.1.

For new construction or major reconstruction of interchanges, 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 (RRR), 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 and storage lengths, and signal phasing. In addition it is difficult to provide proper signing and delineation. Whenever it becomes necessary to locate a ramp terminal close to an intersection, the District Traffic Branch should be consulted regarding the requirement for signing, delineation and signal phasing.

(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

Figure 504.3B
Typical Freeway Entrance Loop Ramp
With 1-Lane Ramp Meter

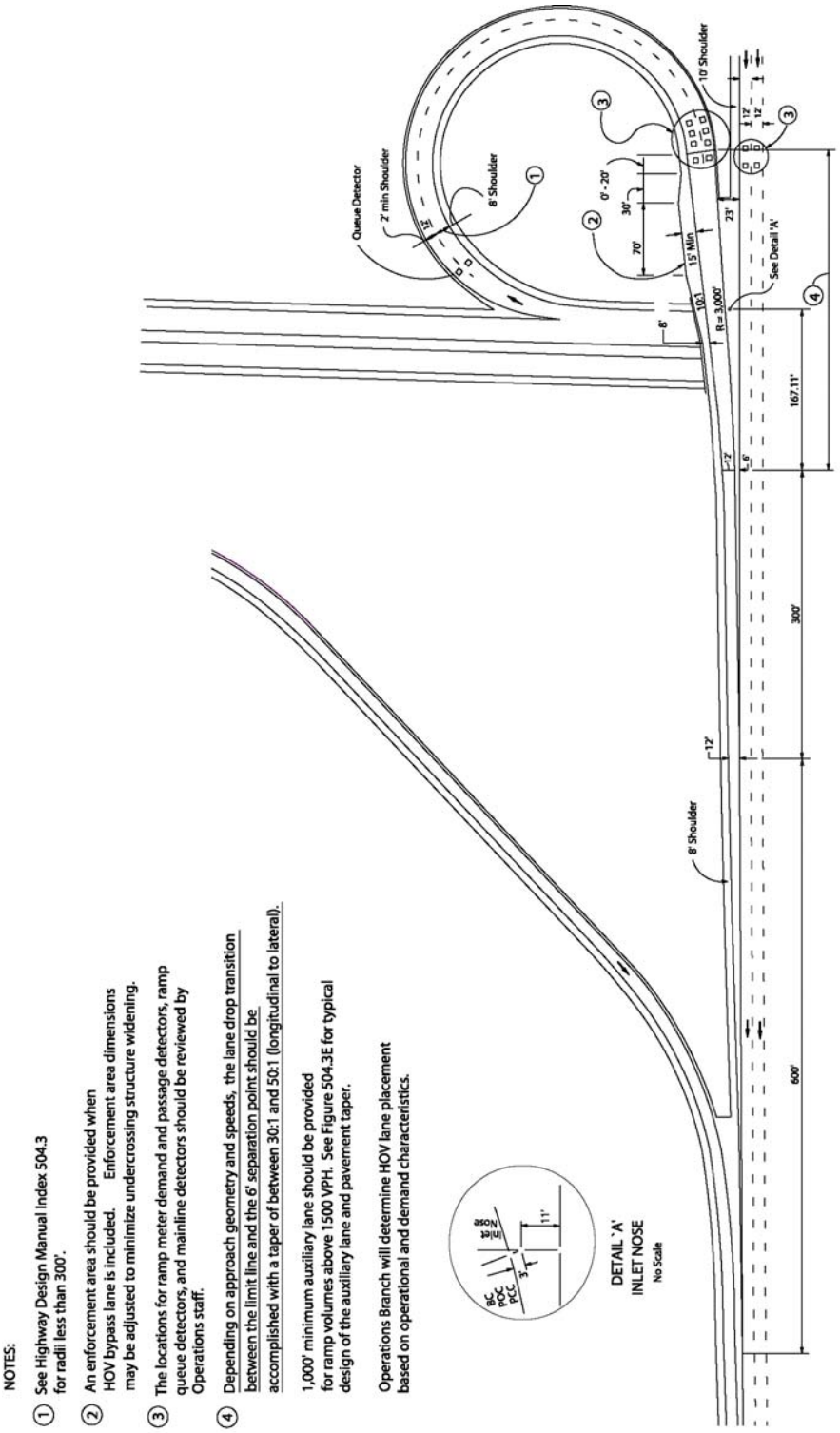


See the MUTCD and California Supplement for signing and striping typicals.

NOTES:

1. See Highway Design Manual Index 504.3 for radii less than 300'.
2. Location for graded enforcement area to be determined by Operations Staff.
3. 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.

Figure 504.3C
Typical Freeway Entrance Loop Ramp
With 2-Lane Ramp Meter



NOTES:

1. See Highway Design Manual Index 504.3 for radii less than 300'.
2. An enforcement area should be provided when HOV bypass lane is included. Enforcement area dimensions may be adjusted to minimize 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 the limit line and the 6' separation point should be accomplished with a taper of between 30:1 and 50:1 (longitudinal to lateral).

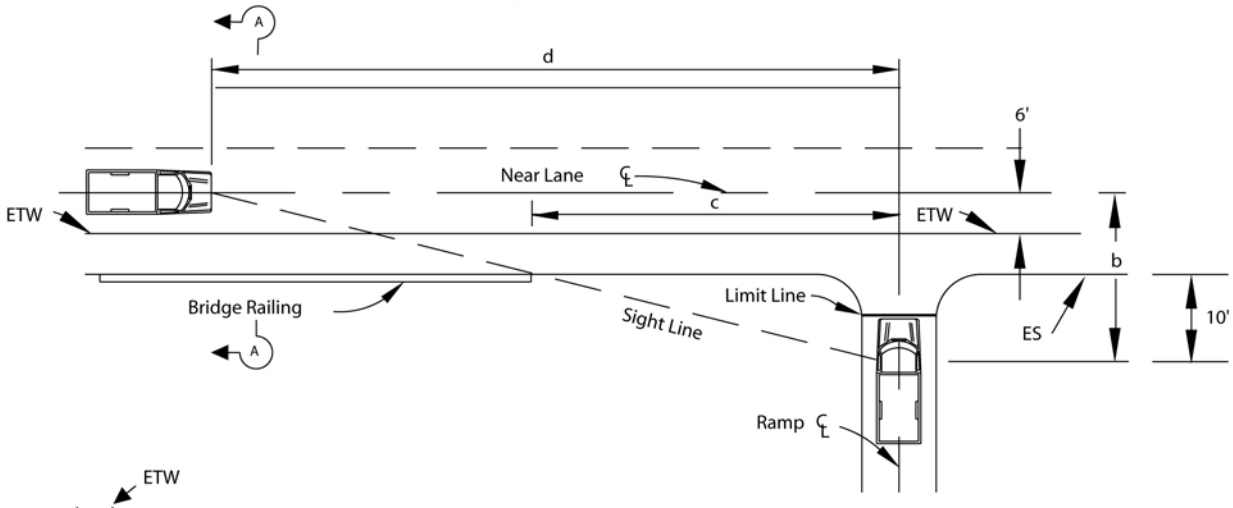
1,000' minimum auxiliary lane should be provided for ramp volumes above 1500 VPH. See Figure 504.3E for typical design of the auxiliary lane and pavement taper.

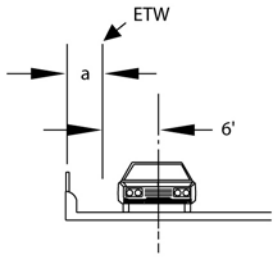
Operations Branch will determine HOV lane placement based on operational and demand characteristics.

See the MUTCD and California Supplement for signing and striping typicals.

Figure 504.3J
Location of Ramp Intersections on the Crossroads

Unsignalized and based on 7.5 second horizontal sight distance criteria





SECTION A - A

$$c = d\left[\frac{b-(a+6')}{b}\right]$$

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.

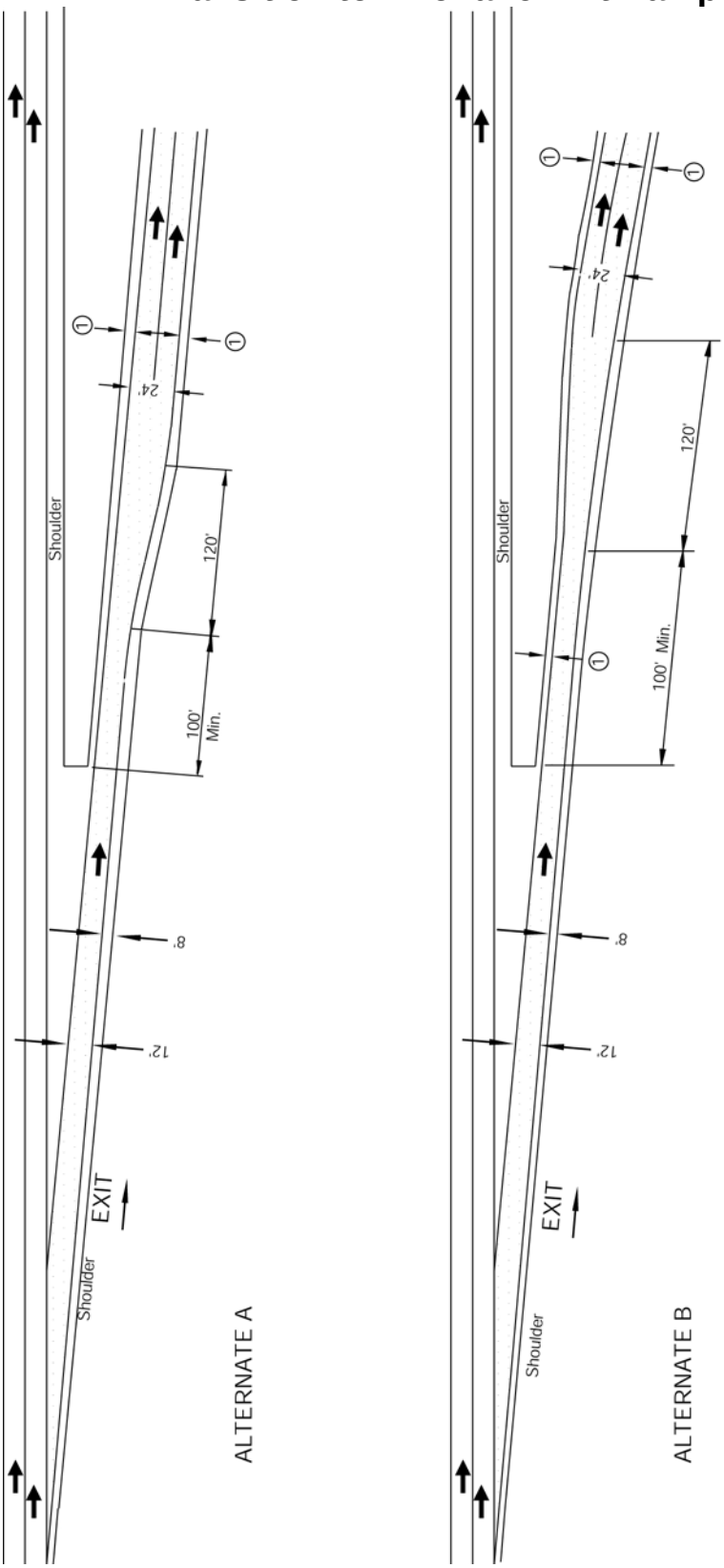
b = Distance from center of near lane to eye of ramp vehicle driver. Ramp driver's eye is assumed to be located 10 ft. from the edge of shoulder, but not less than 15 ft. from the ETW. (Therefore, b = 6' + shoulder width + 10') See Index 405.1.

c = Ramp set back from end of bridge railing.

d = Corner Sight distance along highway from intersection. (See Table above.) Sight distance is measured from a 3½ ft. eye height on the ramp to a 4¼ ft. object height on the crossroad.

V = Anticipated prevailing speed on crossroad.

Figure 504.3K
Transition to Two-lane Exit Ramp

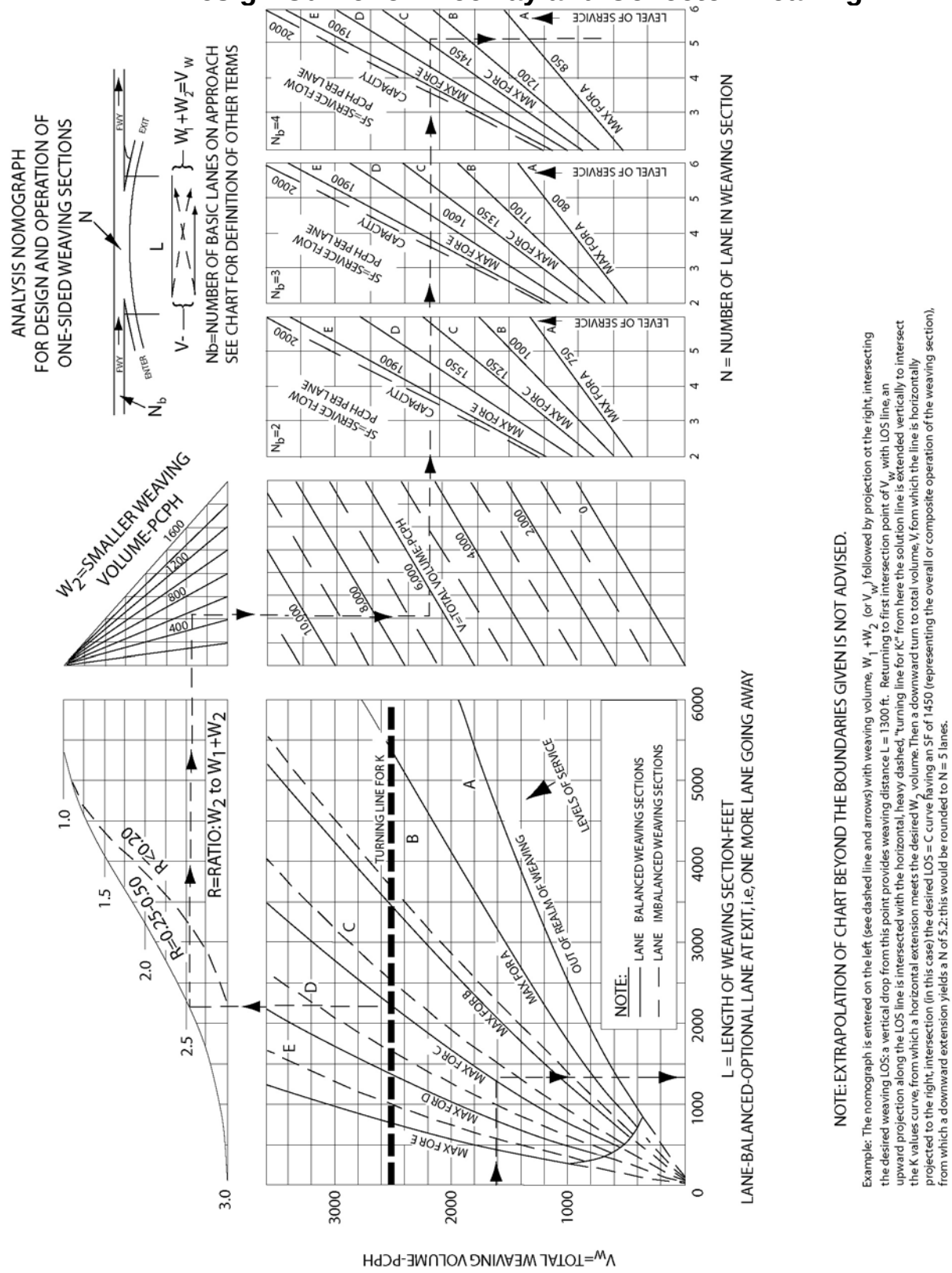


NOTES:

1 See Table 302.1 for shoulder widths. If shoulder reductions occur, see index 206.3(4) for transitions.

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

Figure 504.7A
Design Curve for Freeway and Collector Weaving



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

Example: The nomograph is entered on the left (see dashed line and arrows) with weaving volume, W_1+W_2 (or V_w) followed by projection ot the right, intersecting the desired weaving LOS: a vertical drop from this point provides weaving distance L = 1300 ft. Returning to first intersection point of V_w with LOS line, an upward projection along the LOS line is intersected with the horizontal, heavy dashed, "turning line for K" from here the solution line is extended vertically to intersect the K values curve, from which a horizontal extension meets the desired W_2 volume. Then a downward turn to total volume, V, fom 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.

temperatures); 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 Technical Advisory Guide (MTAG). Note that such strategies are periodically updated.

(2) *Capital Preventive Maintenance (CAPM).* Capital Preventive Maintenance (CAPM) is a program of short-term (<10 years) repair projects agreed to between the Department and FHWA in 1994. Detailed information regarding the CAPM program can be found in the 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. 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 Design Information Bulletin 81, Capital Preventive Maintenance 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).
- Isolated digouts of flexible pavements (up to 20 percent of total project cost).
- 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/or 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 may also include additional items of work such as upgrading other highway appurtenances such as drainage facilities, structures, lighting, signal controllers, and fencing that are failing, worn out or functionally obsolete. Also, unlike pavement rehabilitation and pavement preservation projects, traffic safety enhancements and operational improvements may be added to roadway rehabilitation work if such work is critical or required by FHWA RRR (Resurfacing, Restoration, and Rehabilitation) standards. Other work such as geometric corrections and/or non-capacity increasing operational enhancements may also be added to roadway rehabilitation work if they have a high enough priority. Where conditions warrant, quieter pavement surface treatments and textures could be used to reduce tire/pavement noise in urban areas. In certain cases, the use of quieter pavements may eliminate the need for conventional noise abatement measures such as soundwalls.

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 and other RRR projects may also be found in the Design Information Bulletin, Number 79 - "RRR Design Criteria" 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:

- 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

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 Standards Team 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 Standards Team 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 Chief, Office of Pavement Design, 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 Division of Design 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 structural sections 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 structural sections 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 Pavement Standards Team (PST). The Pavement Standards Team 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 Division of Design, Office of Pavement Design (OPD) for approval. “Special” designs are 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 Division of Design, Office of Pavement Design (OPD) either electronically or with 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).

OPD will act as the Headquarters focal point to obtain concurrence, as required, of PST representatives prior to OPD granting approval of the “special” designs.

606.3 Mechanistic-Empirical Design

Mechanistic-Empirical (ME) Design is currently under development by the Department, FHWA,

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 its terminal serviceability or a condition that requires pavement rehabilitation, (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.

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.

(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) *Contact.* Contact joints (sometimes called construction or cold joint) are joints between slabs that result when concrete is placed at different times. Contact joints can be transverse or longitudinal and are constructed in all types of rigid pavements. Tie bars are typically used at contact joints to connect the adjoining slabs together so that the contact joint will be tightly closed.

(2) *Weakened Plane.* Longitudinal and transverse weakened plane joints (also known as contraction 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 623.1L
Rigid Pavement Catalog
(High Mountain/High Desert, Type I Subgrade Soil) (1), (2), (3), (4)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.80 JPCP 0.35 LCB	0.85 JPCP 0.35 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 0.35 HMA-A	0.90 JPCP 0.50 AB	0.90 JPCP 0.35 ATPB 0.40 AB
9.5 to 10	0.85 JPCP 0.40 LCB	0.85 JPCP 0.40 HMA-A	0.90 JPCP 0.60 AB	0.90 JPCP 0.35 ATPB 0.40 AB	0.90 JPCP 0.40 LCB	0.90 JPCP 0.40 HMA-A	0.95 JPCP 0.60 AB	0.90 JPCP 0.35 ATPB 0.40 AB
10.5 to 11	0.90 JPCP 0.40 LCB	0.90 JPCP 0.40 HMA-A	0.95 JPCP 0.70 AB		0.95 JPCP 0.40 LCB	0.95 JPCP 0.40 HMA-A	1.00 JPCP 0.70 AB	
11.5 to 12	0.95 JPCP 0.40 LCB	0.95 JPCP 0.40 HMA-A			1.05 JPCP 0.40 LCB	1.05 JPCP 0.40 HMA-A		
12.5 to 13	1.00 JPCP 0.50 LCB	1.05 JPCP 0.50 HMA-A			1.10 JPCP 0.50 LCB	1.15 JPCP 0.50 HMA-A		
13.5 to 14	1.05 JPCP 0.50 LCB	1.10 JPCP 0.50 HMA-A			1.15 JPCP 0.50 LCB	1.20 JPCP 0.50 HMA-A		
14.5 to 15	1.10 JPCP 0.50 LCB	1.15 JPCP 0.50 HMA-A			1.20 JPCP 0.50 LCB	1.25 JPCP 0.50 HMA-A		
15.5 to 16	1.15 JPCP 0.50 LCB	1.20 JPCP 0.50 HMA-A			1.25 JPCP 0.50 LCB	1.30 JPCP 0.50 HMA-A		
16.5 to 17	1.20 JPCP 0.50 LCB	1.25 JPCP 0.50 HMA-A			1.30 JPCP 0.50 LCB	1.35 JPCP 0.50 HMA-A		
> 17	1.25 JPCP 0.50 LCB	1.25 JPCP 0.50 HMA-A			1.35 JPCP 0.50 LCB	1.35 JPCP 0.50 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.

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)

	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 0.35 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 0.35 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 0.40 LCB 0.50 AS	0.85 JPCP 0.40 HMA-A 0.50 AS	0.90 JPCP 1.00 AB	0.90 JPCP 0.35 ATPB 0.80 AB	0.90 JPCP 0.40 LCB 0.50 AS	0.90 JPCP 0.40 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 0.40 LCB 0.60 AS	0.90 JPCP 0.40 HMA-A 0.60 AS	0.95 JPCP 1.30 AB		0.95 JPCP 0.40 LCB 0.60 AS	0.95 JPCP 0.40 HMA-A 0.60 AS	1.00 JPCP 1.30 AB	
11.5 to 12	0.95 JPCP 0.40 LCB 0.60 AS	0.95 JPCP 0.40 HMA-A 0.60 AS			1.05 JPCP 0.40 LCB 0.60 AS	1.05 JPCP 0.40 HMA-A 0.60 AS		
12.5 to 13	1.00 JPCP 0.50 LCB 0.70 AS	1.05 JPCP 0.50 HMA-A 0.70 AS			1.10 JPCP 0.50 LCB 0.70 AS	1.15 JPCP 0.50 HMA-A 0.70 AS		
13.5 to 14	1.05 JPCP 0.50 LCB 0.70 AS	1.10 JPCP 0.50 HMA-A 0.70 AS			1.15 JPCP 0.50 LCB 0.70 AS	1.20 JPCP 0.50 HMA-A 0.70 AS		
14.5 to 15	1.10 JPCP 0.50 LCB 0.70 AS	1.15 JPCP 0.50 HMA-A 0.70 AS			1.20 JPCP 0.50 LCB 0.70 AS	1.25 JPCP 0.50 HMA-A 0.70 AS		
15.5 to 16	1.15 JPCP 0.50 LCB 0.70 AS	1.20 JPCP 0.50 HMA-A 0.70 AS			1.25 JPCP 0.50 LCB 0.70 AS	1.30 JPCP 0.50 HMA-A 0.70 AS		
16.5 to 17	1.20 JPCP 0.50 LCB 0.70 AS	1.25 JPCP 0.50 HMA-A 0.70 AS			1.30 JPCP 0.50 LCB 0.70 AS	1.35 JPCP 0.50 HMA-A 0.70 AS		
> 17	1.25 JPCP 0.50 LCB 0.70 AS	1.25 JPCP 0.50 HMA-A 0.70 AS			1.35 JPCP 0.50 LCB 0.70 AS	1.35 JPCP 0.50 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.

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

623.2 Mechanistic-Empirical Method

For information on Mechanistic-Empirical Design application and requirements, see Index 606.3.

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 pavement 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 asphalt overlay. The minimum standard thicknesses for a 20-year design life using this strategy are found in Table 625.1.

Table 625.1

Minimum Standard Thicknesses for Crack, Seat, and Asphalt Overlay(1)

TI <12.0	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	0.50′ HMA SAMI-F or SAMI-R 0.10′ HMA (LC)	0.20′ RHMA-G 0.15′ HMA SAMI-F or SAMI-R 0.10′ HMA (LC)

Notes:

(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)
SAMI-F = Stress Absorbing Membrane Interlayer (Fabric)
SAMI-R = Stress Absorbing Membrane Interlayer (Rubberized)

Note that there are currently no standard crack, seat, nor asphalt 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 Pavement Design 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) Asphalt overlay. 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 are allowed to use the shoulder.

(3) *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 and patched; spalls repaired; and broken slabs or punchouts replaced. Existing thermoplastic traffic stripes and raised 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.

(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.

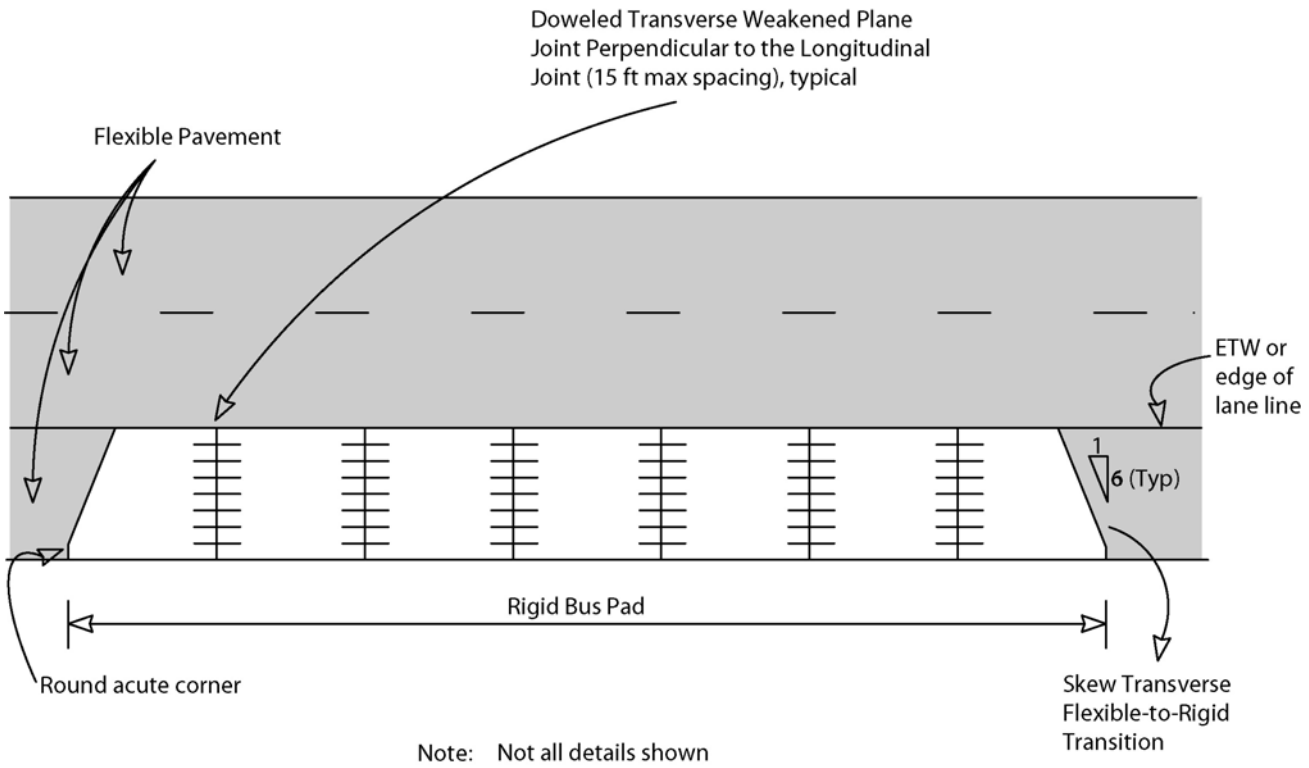
(3) *Bus Pads.* Bus pads are subjected to similar stresses as intersections; however, it is not practical to engineer rigid bus pads according to the Traffic Index, or according to bus counts. The minimum pavement structure for bus pads should be 0.85 foot JPCP with dowel bars at transverse joints on 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 bus 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 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



(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.

Table 632.1

Asphalt Binder Grade

Binder / Climatic Region	Conventional Hot Mixed Asphalt				Rubberized Asphalt
	Dense Graded HMA		Open Graded		Base Stock for Gap and Open Graded
	Typical	Special[1]	Placement Temperature		
			> 70°F	≤ 70°F	
South Coast Central Coast Inland Valleys	PG 64-10	PG 70-10 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[2]	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 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.

(2) PG 64-28 may be specified when specifically requested by the District Materials Engineer.

(3) Consult the District Materials Engineer for which binder grade to use.

Table 663.1A

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 on 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 asphalt	8.05 Btu-in/h-ft^2-°F
Heat capacity asphalt	0.23 Btu/lbm-°F
Base erodibility index (1)	2
LCB 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	43,500 psi/29,000 psi
Plasticity Index	1
Passing No. 200	3%
Passing No. 4	20%
D_{60}	0.315 inch
Base erodibility index(1)	4

Note:

(1) 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 663.1B
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
Hot Mix Asphalt Base	HMAB	NA	(2)
Lime Treated Subbase	LTS	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)

672.3 Guardrails

The extension of the structure approach and sleeper slabs across the full width of the outside shoulder creates a conflict between the outside edge of these slabs and the standard horizontal positioning of some guardrail posts. Spacers are attached to the posts that conflict with the structure approach and sleeper slabs to move the postholes outside the edge of shoulder without changing the standard alignment of the guardrail. These details are covered by DES Standard Details and by the Standard Plans.

672.4 Barriers

On new construction, the structure approach slab extends laterally to coincide with the edge of structure. Any concrete barriers next to the structure approach slab will therefore need to be placed on top of the structure approach slab and part of the responsibilities of the structures engineer. The PE should coordinate with structure engineers to coordinate the limits and responsibility for barriers.

672.5 Structural Approach System Drainage

(1) *Pavement Drainage*. Figure 671.1 shows the components of the positive structural drainage system. Filter fabric should be placed on the grading plane to minimize contamination of the treated permeable base (TPB) for all types of structure approach systems. The plastic pipe shall have a proper outlet to avoid erosion of the structure approach embankment. On all new construction projects, regardless of the type of structure approach slab, provisions for positive drainage of the approach system should be incorporated into the design, see Structures Design Standard Details for requirements. The PE or the District Hydraulics Engineer are responsible for all drainage considerations of the roadway while DES, Structures Design (DES-SD) is responsible for structure related drainage. DES-SD is responsible for engineering of both the approach slab and the drainage system, which normally exits through the wingwall. The highway engineer designs the collection and disposal system, which begins on the outside face of the wingwall.

(2) *Surface Drainage.* Roadway surface drainage should be intercepted before reaching the approach/sleeper slab. The objective is to keep water away from the structure approach embankment. The surface water, once collected, should be discharged at locations where it will not create erosion. Refer to Chapter 831 for more information.

Topic 673 - Structure Approach Slab Rehabilitation Considerations

673.1 Approach Slab Replacement

Replacement of a structural approach slab consists of removing the existing pavement, approach slab, underlying base and subsealing material (if applicable) and then replacing with an appropriate type of structure approach system. Depending on the thickness of the existing surface and base layers to be removed, the minimum 1-foot approach slab thickness may have to be increased. PE needs to make sure the structure engineer addresses this in their reports, plans, and specifications.

673.2 Structure Approach Slab Drainage

Typical details for providing positive drainage of a full-width structure approach system are shown in Figure 673.2. Cross drains are placed at the abutment backwall and at the transverse joint between the existing pavement and the structure approach slab by the structure engineer. A collector/outlet system is placed adjacent to the wingwall at the low side of pavement. The collected water is carried away from the structure approach slab at a location where it will not cause erosion. The PE is responsible for the engineering of the outlet for the structure approach slab drainage. Storm Water Best Management Practices should be considered.

Storm water guidelines are available on the Division of Design, Storm Water website.

The structure approach slab edge details to prevent entry of water at the barrier rail face apply when the wingwalls and/or bridge barrier railing are not being reconstructed.

673.3 Pavement Details

Special pavement details are necessary when structure approach slabs will be replaced in conjunction with the crack, seat, and overlay pavement rehabilitation strategy for rigid pavement. Figure 673.3, which is applicable to full-width slab replacement, illustrates a method of transitioning from a 0.35 foot flexible pavement overlay thickness to a minimum 0.15 foot final flexible overlay thickness. Care should be taken in areas with flat grades to avoid creating a ponding condition at the structure abutment.

Cracking and seating of the existing rigid pavement as well as the geotextile reinforcement fabric should be terminated at the start of the transition from the maximum flexible pavement overlay depth.

Flexible pavement overlays should not be placed on structure decks and approach slabs without the concurrence of Structures Maintenance and Investigations (SMI). If an overlay is needed, SMI will provide the recommended strategy. If another strategy, such as polyester concrete is used, the details will be provided by either SMI or Office of Structure Design (OSD).

673.4 Traffic Handling

Traffic handling considerations typically preclude full-width construction procedures. Structure approach rehabilitation is therefore usually done under traffic control conditions, which require partial-width construction.

District Division of Traffic Operations should be consulted for guidance on lane closures and traffic handling.

When developing traffic handling plans for structure approach slabs, where replacing markings is necessary, and where there is a need to maintain traffic during construction, the Engineer should be aware that pavement joint should not be located underneath any of the wheel paths.

- Slow moving equipment could be kept off the freeway.
- Site not accessible to equipment from the freeway.
- Gates necessary for access to facilities outside the freeway right of way that cannot be reached from local streets or roads.

(b) Proposals for locked gates to be used by other public agencies or utility companies must be submitted to the Chief, Division of Design for approval. The submittal should give all the facts justifying approval and comparisons with alternate solutions.

Criteria for justification are generally the same as for gates used exclusively by highway maintenance forces except for parking. Safe and adequate parking is a necessary part of the solution to access by other agencies.

Locked gates to be used by non-utility entities require FHWA approval under any of the following circumstances:

- The gate is on an Interstate route.
- Federal-aid funds participated in the cost of right of way.
- Federal-aid funds participated or may participate in the cost of construction.

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 will be submitted separately to FHWA by the Division of Design after approval by the Chief, Division of Design.

701.3 Fences on Other Highways

(1) Policy. The State will construct or pay the cost of fences on private property only as a right of way consideration to mitigate damages. State 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.

This policy applies to all private as well as public lands.

(2) Types of Fences. Only Type BW and Type WM fences on either metal or wood posts are to be constructed by the State on highways other than freeways and expressways.

(3) Location of Fences. Fences on other highways are placed along the right of way line inside the abutting property.

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 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

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 the level of precise mathematical expression. All hydrologic analysis methods, whether deterministic or statistical, are based on the information available. A common problem faced by the highway design engineer is that there may be insufficient flow data, and often 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 the 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 highway, ease and economy of maintenance, engineering judgement, 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.

Floodplain Encroachment. An action within the limits of the base flood plain.

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.

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.

Freeboard. (1) The vertical distance between the level of the water surface 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, floating debris, or any other condition or emergency, without overtopping the structure.

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.

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. $F_r = V/(gy)^{1/2}$

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.

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.

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.

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.

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.

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.

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.

Hyetograph. Graphical representation of rainfall intensity against time.

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.

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 is that they should pass a 2% probability flood (50-year). Freeboard, vertical clearance between the lowest structural member and the water surface elevation of the design flood, sufficient to pass the 1% probability flood 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.

The effects of a channel change can vary greatly depending on the site conditions. For example certain streams may have a great tolerance to changes, whereas with others, small changes may have significant impacts. When potentially detrimental effects can be foreseen, plans should be developed to mitigate the effects to within tolerable limits.

865.2 Design Considerations

Channel changes should be designed with extreme caution. Careful study of the stream characteristics upstream and downstream as well as within the channel change area is required to achieve a safe and effective design.

Channel changes usually decrease the surface roughness and increase the 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.

Modification to a natural channel may reduce the available cover for fish and other wildlife in and around the water and eliminate the natural food supplies available in the old channel. The design of new channels involving rivers or streams supporting fish or wildlife must be coordinated with the Department of Fish and Game in accordance with current practices.

On intermittent streams the problems are generally erosion or silting. For a discussion of temporary measures to be made a part of the contract, see Index 110.2.

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. This is most likely to occur in arid or semi-arid locations.

Topic 866 - Freeboard Considerations

866.1 General

Freeboard is the extra height of lining above the design depth where overflow is predicted to cause damage. Freeboard allowances will vary with each situation.

When the possibility of damage is slight or non-existent, or where the type of facility is minor, freeboard need not be provided.

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 866.2

Table 866.2

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.

$$H_w = 0.25\, d_c(1 - 11.1\,(S/S_c - 1)^2)$$

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.

Supercritical flow around a curve will cause the water to rise alternately on the outside and inside of the curves due to cross waves. This cross wave pattern may persist for a considerable distance downstream. Extra height of lining must be provided on both sides of the channel.

The heights required by this superelevation of the water surface can be computed by the following Soil Conservation Service (SCS) formulas:

- Rectangular Channels.

Subcritical flow $$E = \frac{3V^2 b}{4gr}$$

Supercritical flow $$E = \frac{1.2V^2 b}{gr}$$

$$Ø = \cos^{-1}\left(\frac{r - \frac{b}{2}}{r + \frac{b}{2}} \cos B\right) - B$$

- Trapezoidal Channels.

Subcritical flow $$E = \frac{V^2(b + 2Kd)}{2(gr - 2KV^2)}$$

Supercritical flow $$E = \frac{V^2(b + 2Kd)}{gr - 2KV^2}$$

Where E = Maximum height of water surface in feet above depth "d".

V = Average velocity for the flow cross section in feet per second at entrance to curve.

b = Width of rectangular channel or bottom width of trapezoidal channel in feet.

g = Acceleration of gravity
= 32.2 feet per second squared.

r = Radius of channel centerline in feet.

K = Cotangent of bank slope.

d = Depth of flow in feet for straight alignment at entrance to curve.

Ø = Central angle of curve from B.C. to point of beginning of zone of maximum depth in degrees.

B = Wave angle in degrees, defined as:

$$\sin B = \frac{(gd)^{1/2}}{V}$$

Figure 873.3A

Nomograph of Stream-Bank Rock Slope Protection

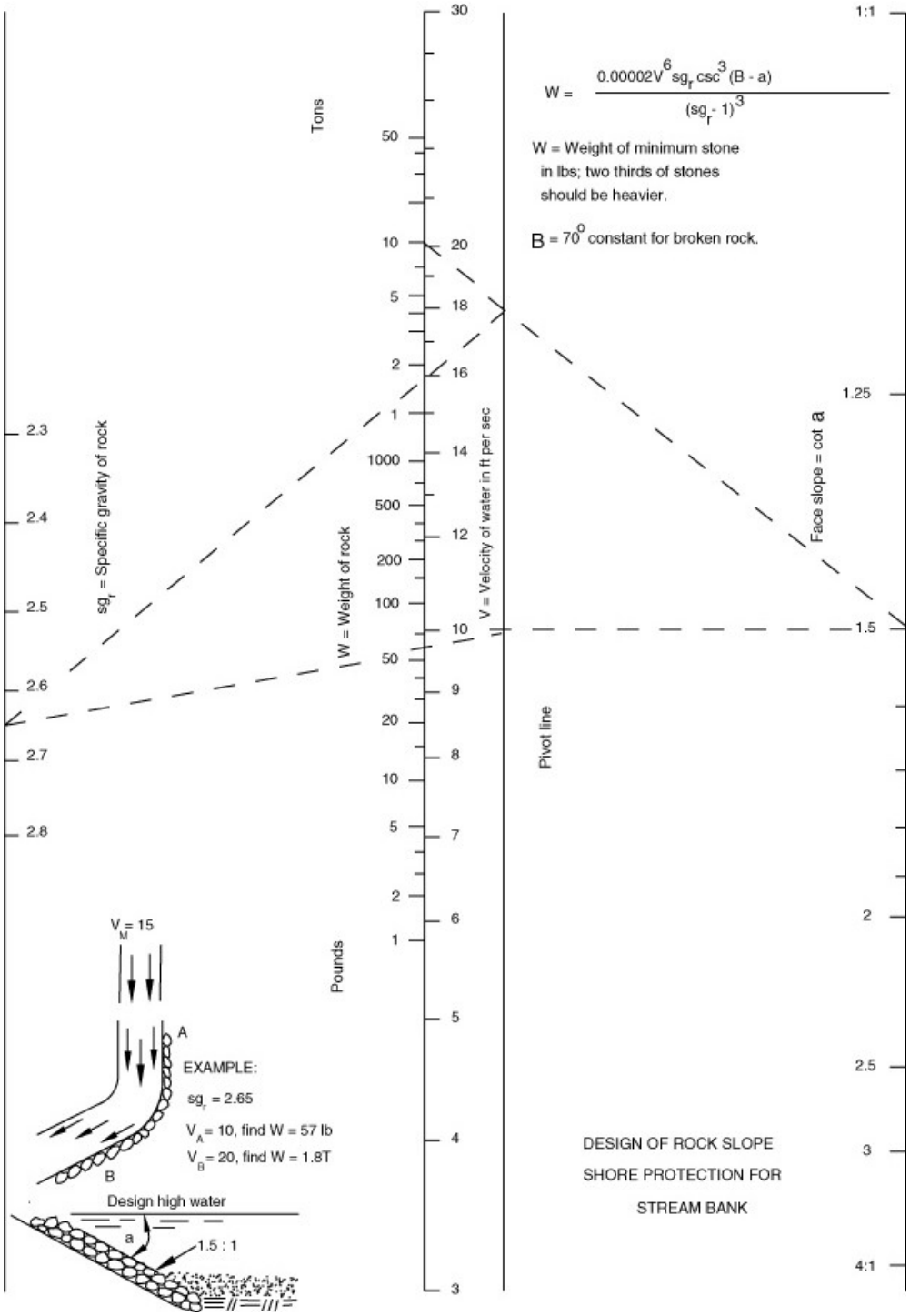
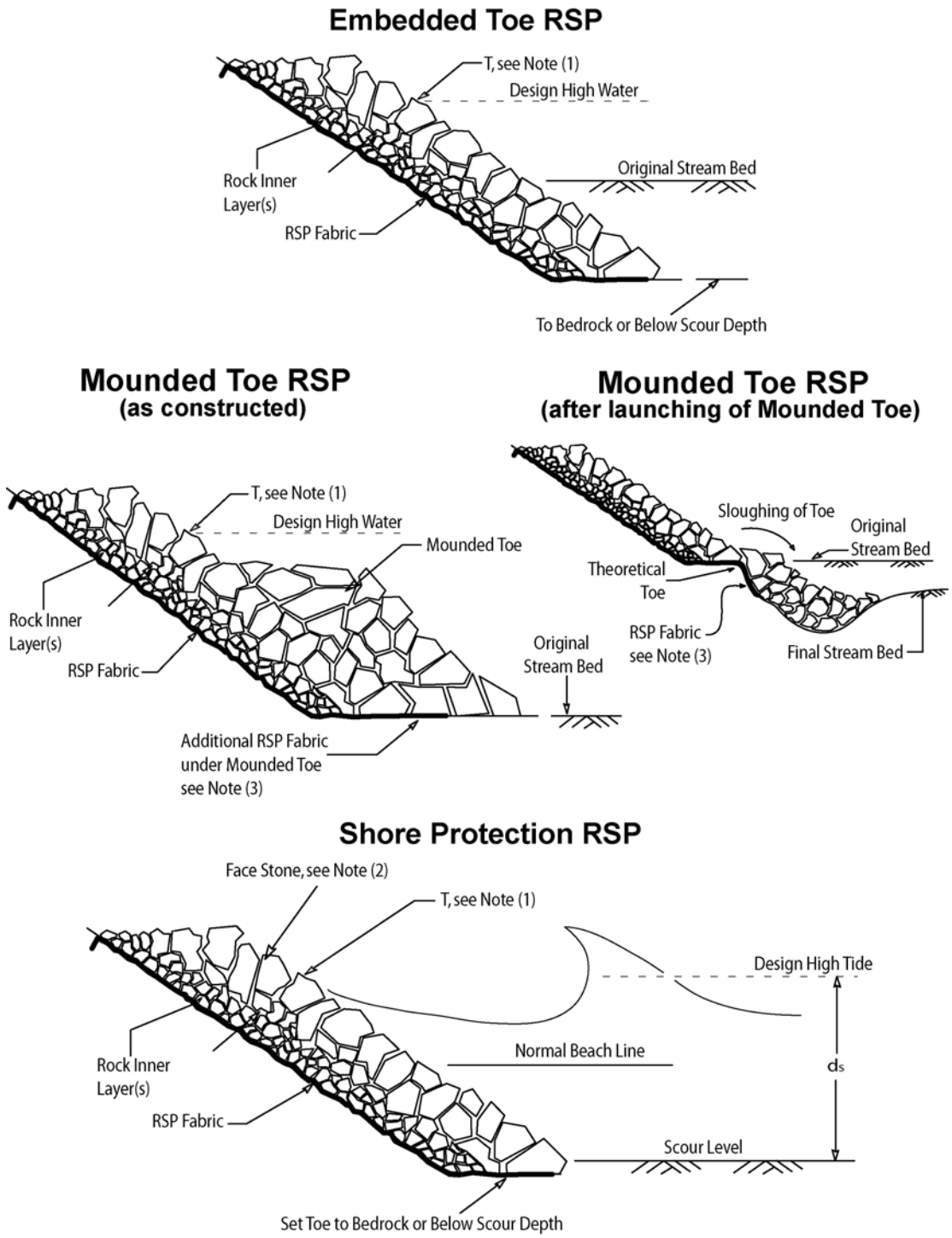


Figure 873.3C
Rock Slope Protection



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.

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 good 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. At least one drinking fountain at each rest area unit must be wheelchair accessible. 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.

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. The Division of Design is responsible for geometric approval. For deviations for this guidance, see Index 82.1. 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 current Americans with Disabilities Act.

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) *Accessibility*. A site must be accessible from a State highway or intersecting road. A site must have adequate sight distance for safe access.

(4) *Accommodation*. A site must be of adequate size to accommodate the necessary features and facilities. However, development of a site shall not detract from the scenic quality of the area. Adequate space should be available for earth mounding and planting to minimize the visual impact of larger facilities. Adequate space for future expansion is desirable.

904.3 Design Features and Facilities

(1) *Road Connections*. The design of connections to vista points should be in accordance with Index 107.1. **Vista points designed for freeways shall have standard freeway exit and entrance ramps (see Chapter 500).**

(2) *Parking*. Paved parking areas should be provided. 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. 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).

(3) *Pedestrian Areas*. Vista points should provide a safe place where motorists can observe the view from outside their vehicles. Walkways may be provided within the viewing area. This space must be accessible to the handicapped and inaccessible to vehicles.

(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. 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.

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 college 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 safe bicycling on existing streets. This can be accomplished by reducing the number of lanes, reducing lane width, or prohibiting 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 controlled by delineation, special efforts should be made to assure that high levels of service are provided with these lanes.

In selecting appropriate streets for bike lanes, location criteria discussed in the next section should be considered.

(4) *Class III Bikeway (Bike Route)*. Bike routes are shared facilities which serve either to:

(a) Provide continuity to other bicycle facilities (usually Class II bikeways); or

(b) Designate preferred routes through high demand corridors.

As with bike lanes, designation of bike routes should indicate to bicyclists that there are particular advantages to using these routes as compared with alternative routes. This means that responsible agencies have taken actions to assure that these routes are suitable as shared routes and will be maintained in a manner consistent with the needs of bicyclists. Normally, bike routes are shared with motor vehicles. The use of sidewalks as Class III bikeways is strongly discouraged.

It is emphasized that the designation of bikeways as Class I, II and III should not be construed as a hierarchy of bikeways; that one is better than the other. Each class of bikeway has its appropriate application.

In selecting the proper facility, an overriding concern is to assure that the proposed facility will not encourage or require bicyclists or motorists to operate in a manner that is inconsistent with the rules of the road.

An important consideration in selecting the type of facility is continuity. Alternating segments of Class I and Class II (or Class III) bikeways along a route are generally incompatible, as street crossings by bicyclists are required when the route changes character. Also, wrong-way bicycle travel will occur on the street beyond the ends of bike paths because of the inconvenience of having to cross the street.

Topic 1003 - Design Criteria

1003.1 Class I Bikeways

Class I bikeways (bike paths) are facilities with exclusive right of way, with cross flows by motorists minimized. Section 890.4 of the Streets and Highways Code describes Class I bikeways as serving "the exclusive use of bicycles and pedestrians". However, experience has shown that if significant pedestrian use is anticipated, separate facilities for pedestrians are necessary to minimize conflicts. Dual use by pedestrians and bicycles is undesirable, and the two should be separated wherever possible.

Sidewalk facilities are not considered Class I facilities because they are primarily intended to serve pedestrians, generally cannot meet the design standards for Class I bikeways, and do not minimize motorist cross flows. See Index 1003.3 for discussion relative to sidewalk bikeways.

By State law, motorized bicycles ("mopeds") are prohibited on bike paths unless authorized by ordinance or approval of the agency having jurisdiction over the path. Likewise, all motor vehicles are prohibited from bike paths. These prohibitions can be strengthened by signing.

(1) *Widths.* **The minimum paved width for a two-way bike path shall be 8 feet. The minimum paved width for a one-way bike path shall be 5 feet. A minimum 2-foot wide graded area shall be provided adjacent to the pavement (see Figure 1003.1A).** A 3-foot graded area is recommended to provide clearance from poles, trees, walls, fences, guardrails, or other lateral obstructions. A wider graded area can also serve as a jogging path. Where the paved width is wider than the minimum required, the graded area may be reduced accordingly; however, the graded area is a desirable feature regardless of the paved width. Development of a one-way bike path should be undertaken only after careful consideration due to the problems of enforcing one-way operation and the difficulties in maintaining a path of restricted width.

Where heavy bicycle volumes are anticipated and/or significant pedestrian traffic is expected, the paved width of a two-way path should be greater than 8-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, necessitating more width for safe use.

Experience has shown that paved paths less than 12 feet wide sometimes break up along the edge as a result of loads from maintenance vehicles.

Where equestrians are expected, a separate facility should be provided.

(2) *Clearance to Obstructions.* **A minimum 2-foot horizontal clearance to obstructions shall be provided adjacent to the pavement (see Figure 1003.1A).** A 3-foot clearance is recommended. Where the paved width is wider than the minimum required, the clearance may be reduced accordingly; however, an adequate clearance is desirable regardless of the paved width. If a wide path is paved contiguous with a continuous fixed object (e.g., block wall), 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 on structures between railings shall be not less than 8 feet.** It is desirable that the clear width of structures be equal to the minimum clear width of the path (i.e., 12 feet).

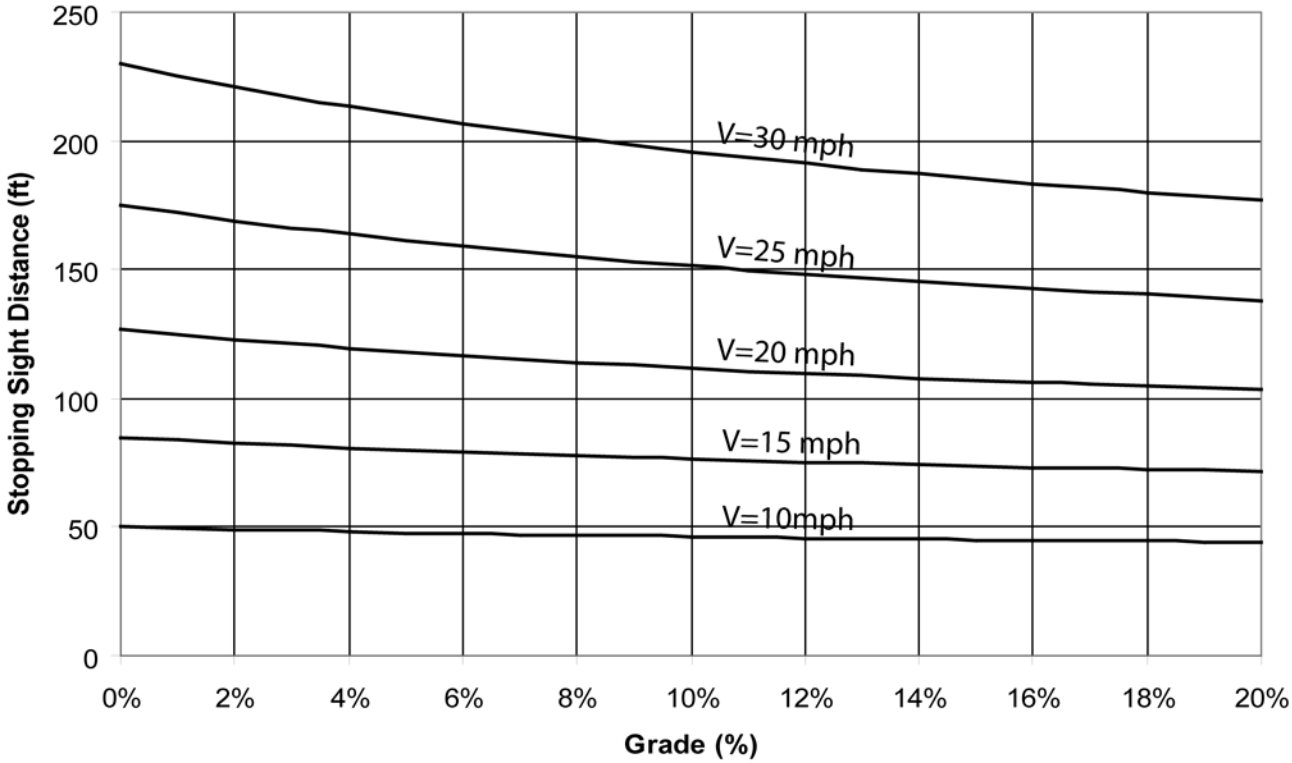
The vertical clearance to obstructions across the clear width of the path shall be a minimum of 8 feet. Where practical, a vertical clearance of 10 feet is desirable.

(3) *Signing and Delineation*. For application and placement of signs, see the California Manual on Uniform Traffic Control Devices (California MUTCD), Section 9B.01. For pavement marking guidance, see the California MUTCD, Section 9C.03.

(4) *Intersections with Highways*. Intersections are a prime consideration in bike path design. If alternate locations for a bike path are available, the one with the most favorable intersection conditions should be selected.

Figure 1003.1E

Stopping Sight Distance – Ascending Grade



$$S = \frac{V^2}{30(f + G)} + 3.67V$$

Where : S = Stopping sight distance (ft)

V = Velocity (mph)

f = Coefficient of friction (use 0.25)

G = Grade (ft/ft) rise/run

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, and because of the serious consequences of a head on bicycle accident, 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, consideration should be given to widening the path through the curve, installing a yellow center line, installing a curve warning sign, or some combination of these alternatives.

(12) *Grades*. Bike paths generally attract less skilled bicyclists, so it is important to avoid steep grades in their design. Bicyclists not physically conditioned will be unable to negotiate long, steep uphill grades. Since novice bicyclists often ride poorly maintained bicycles, long downgrades can cause problems. For these reasons, bike paths with long, steep grades will generally receive very little use. The maximum grade rate recommended for bike paths is 5 percent. It is desirable that sustained grades be limited to 2 percent if a wide range of riders is to be accommodated. Steeper grades can be tolerated for short segments (e.g., up to about 500 feet). Where steeper grades are necessitated, the design speed should be increased and additional width should be provided for maneuverability.

(13) *Pavement Structure*. The pavement structure of a bike path should be designed in the same manner as a highway, with consideration given to the quality of the basement soil and the anticipated loads the bikeway will experience. It is important to construct and maintain a smooth riding surface with skid resistant qualities. Principal loads will normally be from maintenance and emergency vehicles. Expansive soil should be given special consideration and will probably require a special pavement structure. A minimum pavement thickness of 2 inches of Hot Mix Asphalt (HMA) is recommended. HMA (as described in Department of Transportation Standard Specifications), with ½ inch maximum aggregate and medium grading is recommended. Consideration should be given to increasing the asphalt content to provide increased pavement life. Consideration should also be given to sterilization of basement soil to preclude possible weed growth through the pavement.

At unpaved highway or driveway crossings of bicycle paths, the highway or driveway should be paved a minimum of 10 feet on each side of the crossing to reduce the amount of gravel being scattered along the path by motor vehicles. The pavement structure at the crossing should be adequate to sustain the expected loading at that location.

(14) *Drainage.* For proper drainage, the surface of a bike path should have a cross slope of 2 percent. Sloping in one direction usually simplifies longitudinal drainage design and surface construction, and accordingly is the preferred practice. 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 across 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.

(15) *Barrier Posts*. It may be necessary to install barrier posts at entrances to bike paths to prevent motor vehicles from entering. For barrier post placement, visibility marking, and pavement markings, see the California MUTCD, Section 9C.101(CA).

Generally, barrier configurations that preclude entry by motorcycles present safety and convenience problems for bicyclists. Such devices should be used only where extreme problems are encountered.

(16) Lighting. Fixed-source lighting reduces 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 riding at night is expected, such as bicycle paths serving college students or commuters, and at highway intersections. Lighting should also be considered through underpasses or tunnels, and when nighttime security could be a problem.

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.

1003.2 Class II Bikeways

Class II bikeways (bike lanes) for preferential use by bicycles are established within the paved area of highways. Bike lane pavement markings are intended to promote an orderly flow of traffic, by establishing specific lines of demarcation between areas reserved for bicycles and lanes to be occupied by motor vehicles. This effect is supported by bike lane signs and pavement markings. Bike lane pavement markings can increase bicyclists' confidence that motorists will not stray into their path of travel if they remain within the bike lane. Likewise, with more certainty as to where bicyclists will be, passing motorists are less apt to swerve toward opposing traffic in making certain they will not hit bicyclists.

Class II bike lanes shall be one-way facilities. Two-way bike lanes (or bike paths that are contiguous to the roadway) are not permitted, as such facilities have proved unsatisfactory and promote riding against the flow of motor vehicle traffic.

(1) Widths. Typical Class II bikeway configurations are illustrated in Figure 1003.2A and are described below:

(a) Figure 1003.2A-(1) depicts bike lanes on an urban type curbed street where parking stalls (or continuous parking stripes) are marked. Bike lanes are located between the parking area and the traffic lanes. **As indicated, 5 feet shall be the minimum width of bike lane where parking stalls are marked.** If parking volume is substantial or turnover high, an additional 1 foot to 2-foot of width is desirable.

Bike lanes shall not be placed between the parking area and the curb. Such facilities increase the conflict between bicyclists and opening car doors and reduce visibility at intersections. Also, they prevent bicyclists from leaving the bike lane to turn left and cannot be effectively maintained.

(b) Figure 1003.2A-(2) depicts bike lanes on an urban-type curbed street, where parking is permitted, but without parking stripe or stall marking. Bike lanes are established in conjunction with the parking areas**. As indicated, 11 feet or 12 feet (depending on the type of curb) shall be the minimum width of the bike lane where parking is permitted.** This type of lane is satisfacory where parking is not extensive and where turnover of parked cars is infrequent. However, if parking is substantial, turnover of parked cars is high, truck traffic is substantial, or if vehicle speeds exceed 35 miles per hour, additional width is recommended.

(c) Figure 1003.2A-(3) depicts bike lanes along the outer portions of an urban type curbed street, where parking is prohibited. This is generally the most desirable configuration for bike lanes, as it eliminates potential conflicts resulting from auto parking (e.g., opening car doors). **As indicated, if no gutter exists, the minimum bike lane width shall be 4 feet. With a normal 2-foot gutter, the minimum bike lane width shall be 5 feet.** The intent is to provide a minimum 4 feet wide bike lane, but with at least 3 feet between the traffic lane and the longitudinal joint at the concrete gutter, since the gutter reduces the effective width of the bike lane for two reasons. First, the longitudinal joint may not always be smooth, and may be difficult

to ride along. Secondly, the gutter does not provide a suitable surface for bicycle travel. Where gutters are wide (say, 4 feet), an additional 3 feet must be provided because bicyclists should not be expected to ride in the gutter. Wherever possible, the width of bike lanes should be increased 6 feet to 8 feet to provide for greater safety. Eight-foot bike lanes can also serve as emergency parking areas for disabled vehicles.

Striping bike lanes next to curbs where parking is prohibited only during certain hours shall be done only in conjunction with special signing to designate the hours bike lanes are to be effective. Since the Vehicle Code requires bicyclists to ride in bike lanes where provided (except under certain conditions), proper signing is necessary to inform bicyclists that they are required to ride in bike lanes only during the course of the parking prohibition. This type of bike lane should be considered only if the vast majority of bicycle travel would occur during the hours of the parking prohibition, and only if there is a firm commitment to enforce the parking prohibition. Because of the obvious complications, this type of bike lane is not encouraged for general application.

Figure 1003.2A-(4) depicts bike lanes on a highway without curbs and gutters. This location is in an undeveloped area where infrequent parking is handled off the pavement. This can be accomplished by supplementing the bike lane signing with R25 (park off pavement) signs, or R26 (no parking) signs. **Minimum widths shall be as shown.** Additional width is desirable, particularly where motor vehicle speeds exceed 35 miles per hour

Per Topic 301, the minimum lane width standard is 12 feet. There are situations where it may be desirable to reduce the width of the traffic lanes in order to add or widen bicycle 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, bicycle lane width, sight distance, and the presence of on-street vehicle parking. When vehicle parking is permitted adjacent to a bicycle lane, or on a shoulder where bicycling is not prohibited, reducing the width of the adjacent traffic lane may allow for wider bicycle lanes or shoulders, to provide greater clearance between bicyclists and driver-side doors when opened. Where favorable conditions exist, traffic lanes of 11 feet may be feasible but must be approved per Topic 301.

Bike lanes are not advisable on long, steep downgrades, where bicycle speeds greater than 30 miles per hour are expected. As grades increase, downhill bicycle speeds will increase, which increases the problem of riding near the edge of the roadway. In such situations, bicycle speeds can approach those of motor vehicles, and experienced bicyclists will generally move into the motor vehicle lanes to increase sight distance and maneuverability. If bike lanes are to be marked, additional width should be provided to accommodate higher bicycle speeds.

If the bike lanes are to be located on one-way streets, they should be placed on the right side of the street. Bike lanes on the left side would cause bicyclists and motorists to undertake crossing maneuvers in making left turns onto a two-way street.

(2) *Signing and Pavement Markings*. Details for signing and pavement marking of Class II bikeways are found in the California MUTCD, Section 9C.04.

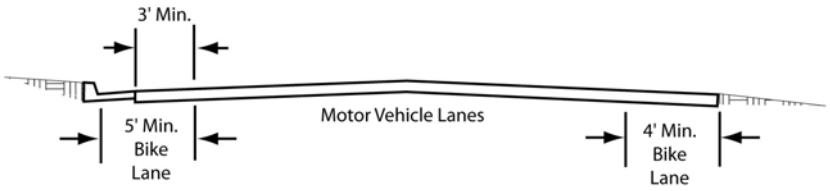
(3) *At-grade Intersection Design*. Most auto/bicycle accidents occur at intersections. For this reason, bikeway design at intersections should be accomplished in a manner that will minimize confusion by motorists and bicyclists, and will permit both to operate in accordance with the normal rules of the road.

Figure 1003.2A
Typical Bike Lane Cross Sections
(On 2-lane or Multilane Highways)

(1) MARKED PARKING

* 13' is recommended where there is substantial parking or turnover of parked cars is high (e.g. commerical areas).

(2) PARKING PERMITTED WITHOUT MARKED PARKING OR STALL



(With Gutter) **(3) PARKING PROHIBITED** (Without Gutter)

(4) TYPICAL ROADWAY IN OUTLYING AREAS PARKING RESTRICTED

Note: For pavement marking guidance, see the California MUTCD, Section 9C.04

Figure 1003.2B illustrates a typical at-grade intersection of multilane streets, with bike lanes on all approaches. Some common movements of motor vehicles and bicycles are shown. A prevalent type of accident involves straight-through bicycle traffic and right-turning motorists. Left-turning bicyclists also have problems, as the bike lane is on the right side of the street, and bicyclists have to cross the path of cars traveling in both directions. Some bicyclists are proficient enough to merge across one or more lanes of traffic, to use the inside lane or left-turn lane. However, there are many who do not feel comfortable making this maneuver. They have the option of making a two-legged left turn by riding along a course similar to that followed by pedestrians, as shown in the diagram. Young children will often prefer to dismount and change directions by walking their bike in the crosswalk.

(4) Interchange Design. As with bikeway design through at-grade intersections, bikeway design through interchanges should be accomplished in a manner that will minimize confusion by motorists and bicyclists. Designers should work closely with the local agency in designing bicycle facilities through interchanges. Local Agencies should carefully select interchange locations which are most suitable for bikeway designations and where the crossing meets applicable design standards. The local agency may have special needs and desires for continuity through interchanges which should be considered in the design process.

For Class II bikeway signing and lane markings, see the California MUTCD, Section 9C.04.

The shoulder width shall not be reduced through the interchange area. The minimum shoulder width shall match the approach roadway shoulder width, but not less than 4 feet or 5 feet if a gutter exists. If the shoulder width is not available, the designated bike lane shall end at the previous local road intersection.

Depending on the intersection angles, either Figure 1003.2C or 1003.2D should also be used for multilane ramp intersections. Additionally, the outside through lane should be widened to 14 feet when feasible. This allows extra room for bicycles to share the through lane with vehicles. The outside shoulder width should not be reduced through the interchange area to accommodate this additional width.

1003.3 Class III Bikeways

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 shared facilities, either with motor vehicles on the street, or with pedestrians on sidewalks, and in either case bicycle usage is secondary. Class III facilities are established by placing Bike Route signs along roadways.

Minimum widths for Class III bikeways are not presented, as the acceptable width is dependent on many factors, including the volume and character of vehicular traffic on the road, typical speeds, vertical and horizontal alignment, sight distance, and parking conditions.

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) An effort has been made to adjust traffic control devices (stop signs, signals) to give greater priority to bicyclists, as compared with alternative streets. This could include placement of bicycle-sensitive detectors on the right-hand portion of the road, where bicyclists are expected to ride.

Figure 1003.2B

Typical Bicycle/Auto Movements at Intersections of Multilane Streets

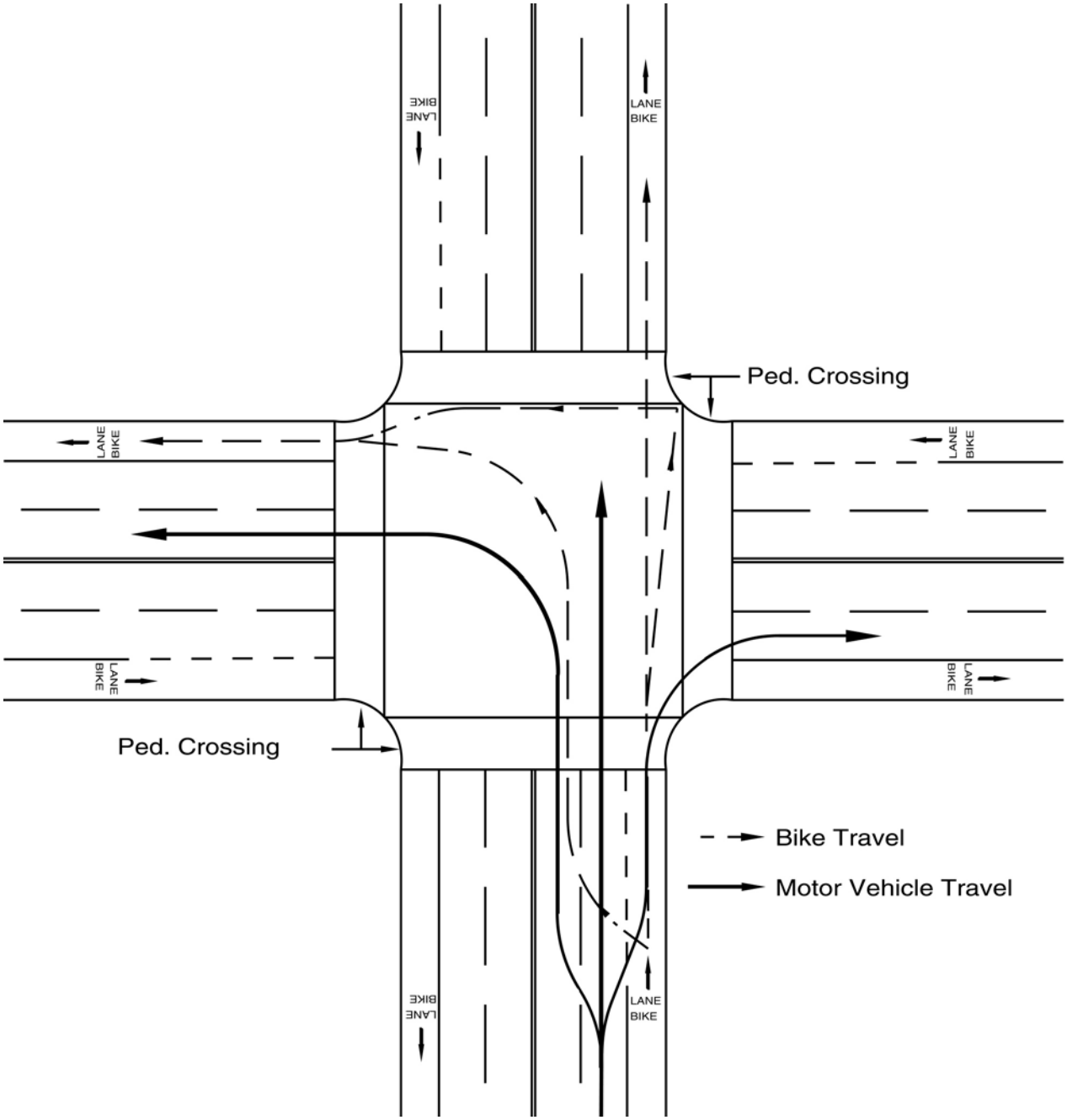
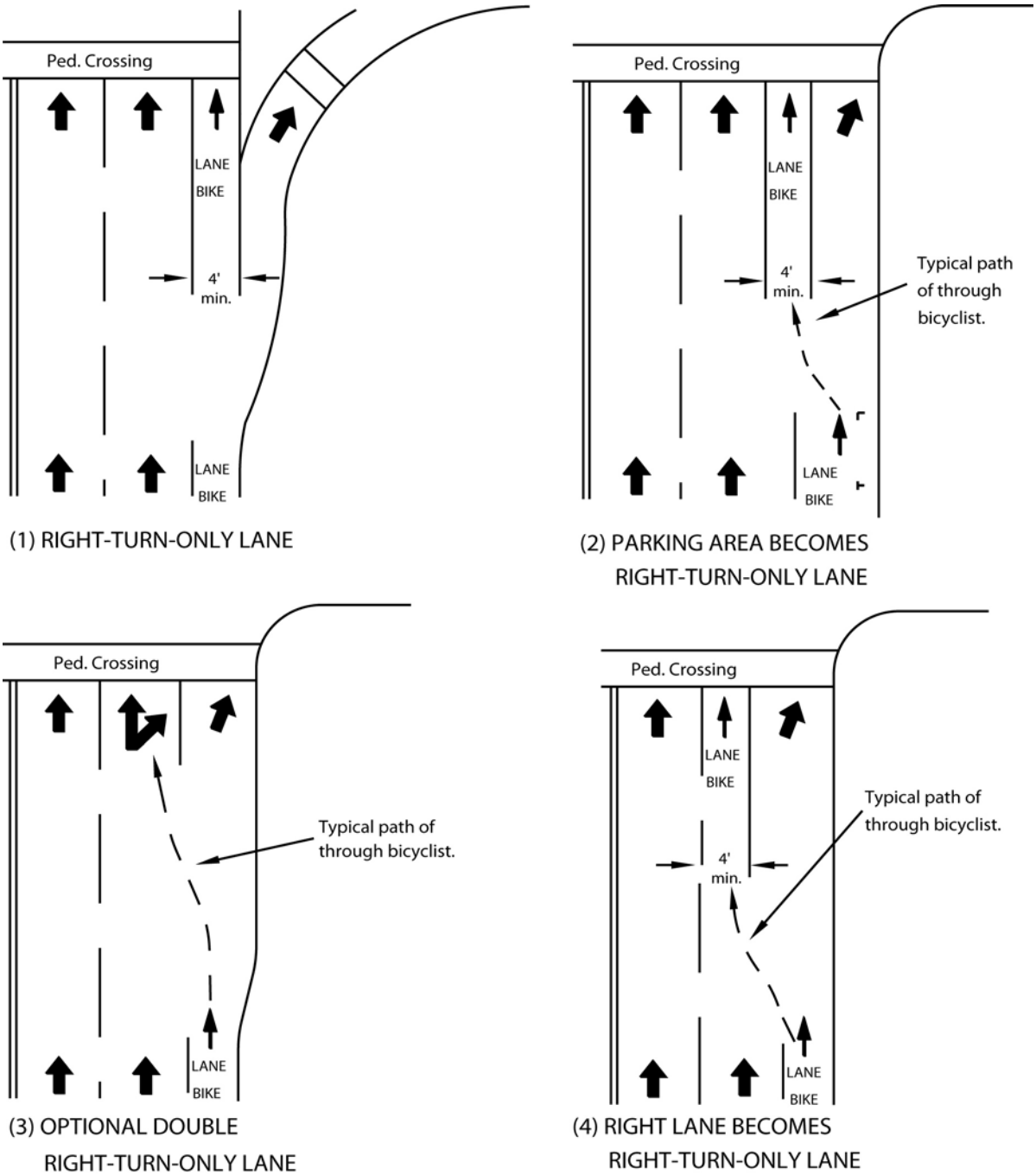
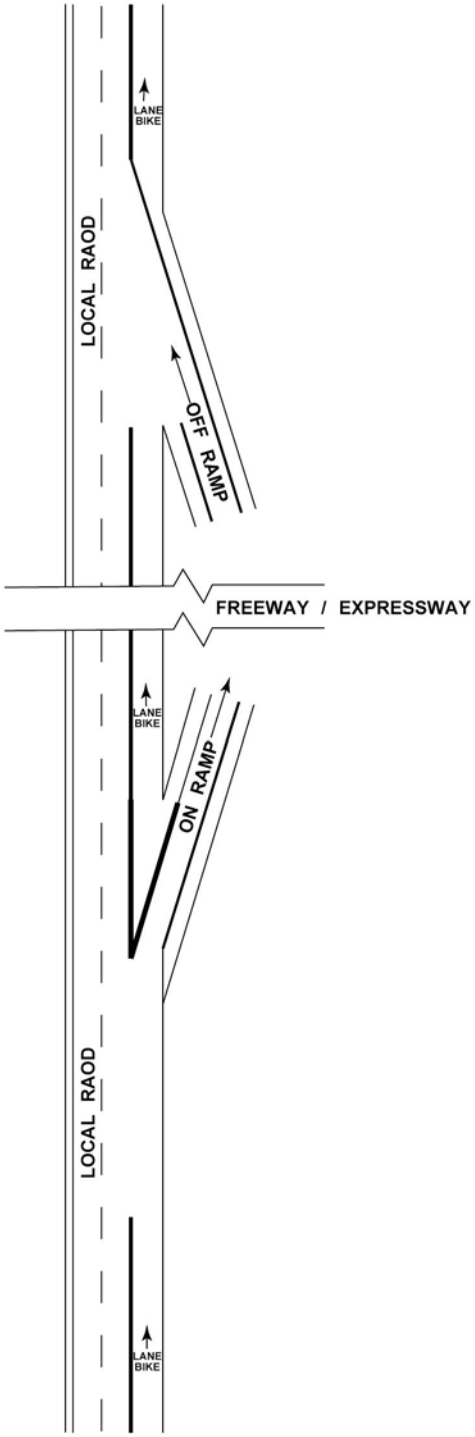


Figure 1003.2C
Bike Lanes Approaching Motorist
Right-turn-only Lane



Note: For bicycle lane markings, see the California MUTCD, Section 9C.04.

Figure 1003.2D
Bike Lanes Through Interchanges



Notes:

1.) See Index 1003.2(4) for additional information.

2.) The shoulder width shall not be reduced through the interchange area. The minimum shoulder width shall match the approach roadway shoulder width, but not less than 4 feet or 5 feet if a gutter exists. If the shoulder width is not available, the designated bike lane shall end at the previous local raod intersection.

3.) See Index 1003.3(4) for information on Bike Routes Through Interchanges.

(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 Bikeway Criteria. In general, the designated use of sidewalks (as a Class III bikeway) for bicycle travel is unsatisfactory.

It is important to recognize that the development of extremely wide sidewalks does not necessarily add to the safety of sidewalk bicycle travel, as wide sidewalks will encourage higher speed bicycle use and can increase potential for conflicts with motor vehicles at intersections, as well as with pedestrians and fixed objects.

Sidewalk bikeways should be considered only under special circumstances, such as:

(a) To provide bikeway continuity along high speed or heavily traveled roadways having inadequate space for bicyclists, and uninterrupted by driveways and intersections for long distances.

(b) On long, narrow bridges. In such cases, ramps should be installed at the sidewalk approaches. If approach bikeways are two-way, sidewalk facilities should also be two-way.

Whenever sidewalk bikeways are established, a special effort should be made to remove unnecessary obstacles. Whenever bicyclists are directed from bike lanes to sidewalks, curb cuts should be flush with the street to assure that bicyclists are not subjected to problems associated with crossing a vertical lip at a flat angle. Also curb cuts at each intersection are necessary. Curb cuts should be wide enough to accommodate adult tricycles and two-wheel bicycle trailers.

In residential areas, sidewalk riding by young children too inexperienced to ride in the street is common. With lower bicycle speeds and lower auto speeds, potential conflicts are somewhat lessened, but still exist. Nevertheless, this type of sidewalk bicycle use is accepted. But it is inappropriate to sign these facilities as bikeways. Bicyclists should not be encouraged (through signing) to ride facilities that are not designed to accommodate bicycle travel.

(3) Destination Signing of Bike Routes. For Bike Route signs to be more functional, supplemental plates may be placed beneath them when located along routes leading to high demand destinations (e.g., "To Downtown"; "To State College"; etc. For typical signing, see the California MUTCD, Figures 9B-5 and 9B-6.

There are instances where it is necessary to sign a route to direct bicyclists to a logical destination, but where the route does not offer any of the above listed bike route features. In such cases, the route should not be signed as a bike route; however, destination signing may be advisable. A typical application of destination signing would be where bicyclists are directed off a highway to bypass a section of freeway. Special signs would be placed to guide bicyclists to the next logical destination. The intent is to direct bicyclists in the same way as motorists would be directed if a highway detour was necessitated.

(4) Interchange Design As with bikeway design through at-grade intersections, bikeway design through interchanges should be accomplished in a manner that will minimize confusion by motorists and bicyclists. Designers should work closely with the local agency in designing bicycle facilities through interchanges. Local Agencies should carefully select interchange locations which are most suitable for bikeway designations and where the crossing meets applicable design standards. The local agency may have special needs and desires for continuity through interchanges which should be considered in the design process.

Within the Interchange area the bike route shall require either an outside lane width of 16-foot or a 12-foot lane and a 4-foot shoulder. If the above width is not available,

the designated bike route shall end at the previous local road intersection.

1003.4 Bicycles on Freeways

In some instances, bicyclists are permitted on freeways. Seldom would a freeway be designated as a bikeway, but it can be opened for use if it meets certain criteria. Essentially, the criteria involve assessing the safety and convenience of the freeway as compared with available alternate routes. However, a freeway should not be opened to bicycle use if it is determined to be incompatible. The Headquarters Traffic Liaisons and the Design Coordinator must approve any proposals to open freeways to bicyclists.

If a suitable alternate route exists, it would normally be unnecessary to open the freeway. However, if the alternate route is unsuitable for bicycle travel the freeway may be a better alternative for bicyclists. In determining the suitability of an alternate route, safety should be the paramount consideration. The following factors should be considered:

- Number of intersections
- Shoulder widths
- Traffic volumes
- Vehicle speeds
- Bus, truck and recreational vehicle volumes
- Grades
- Travel time

When a suitable alternate route does not exist, a freeway shoulder may be considered for bicycle travel. Normally, freeways in urban areas will have characteristics that make it unfeasible to permit bicycle use. In determining if the freeway shoulder is suitable for bicycle travel, the following factors should be considered;

- Shoulder widths
- Bicycle hazards on shoulders (drainage grates, expansion joints, etc.)
- Number and location of entrance/exit ramps
- Traffic volumes on entrance/exit ramps
- Bridge Railing height

When bicyclists are permitted on segments of freeway, it will be necessary to modify and supplement freeway regulatory signs, particularly those at freeway ramp entrances and exits, see the California MUTCD, Section 9B.101(CA).

Where no reasonable alternate route exists within a freeway corridor, the Department should coordinate with local agencies to develop or improve existing routes or provide parallel bikeways within or adjacent to the freeway right of way.

The long term goal is to provide a safe and convenient non-freeway route for bicycle travel.

1003.5 Multipurpose Trails

In some instances, it may be appropriate for agencies to develop multipurpose trails - for hikers, joggers, equestrians, bicyclists, etc. Many of these trails will not be paved and will not meet the standards for Class I bikeways. As such, these facilities should not be signed as bikeways. Rather, they should be designated as multipurpose trails (or similar designation), along with regulatory signing to restrict motor vehicles, as appropriate.

If multipurpose trails are primarily to serve bicycle travel, they should be developed in accordance with standards for Class I bikeways. In general, multipurpose trails are not recommended as high speed transportation facilities for bicyclists because of conflicts between bicyclists and pedestrians. Wherever possible, separate bicycle and pedestrian paths should be provided. If this is not feasible, additional width, signing and pavement markings should be used to minimize conflicts.

It is undesirable to mix mopeds and bicycles on the same facility. In general, mopeds should not be allowed on multipurpose trails because of conflicts with slower moving bicyclists and pedestrians. In some cases where an alternate route for mopeds does not exist, additional width, signing, and pavement markings should be used to minimize conflicts. Increased patrolling by law enforcement personnel is also recommended to enforce speed limits and other rules of the road.

It is usually not desirable to mix horses and bicycle traffic on the same multipurpose trail. Bicyclists are often not aware of the need for slower speeds and additional operating space near horses. Horses

can be startled easily and may be unpredictable if they perceive approaching bicyclists as a danger. In addition, pavement requirements for safe bicycle travel are not suitable for horses. For these reasons, a bridle trail separate from the multipurpose trail is recommended wherever possible.

1003.6 Miscellaneous Bikeway Criteria

The following are miscellaneous bikeway criteria which should be followed to the extent pertinent to Class I, II and III bikeways. Some, by their very nature, will not apply to all classes of bikeway. Many of the criteria are important to consider on any highway where bicycle travel is expected, without regard to whether or not bikeways are established.

(1) Bridges. Bikeways on highway bridges must be carefully coordinated with approach bikeways to make sure that all elements are compatible. For example, bicycle traffic bound in opposite directions is best accommodated by bike lanes on each side of a highway. In such cases, a two-way bike path on one side of a bridge would normally be inappropriate, as one direction of bicycle traffic would be required to cross the highway at grade twice to get to and from the bridge bike path. Because of the inconvenience, many bicyclists will be encouraged to ride on the wrong side of the highway beyond the bridge termini.

The following criteria apply to a two-way bike path on one side of a highway bridge:

(a) The bikeway approach to the bridge should be by way of a separate two-way facility for the reason explained above.

(b) **A physical separation, such as a chain link fence or railing, shall be provided to offset the adverse effects of having bicycles traveling against motor vehicle traffic.** The physical separation should be designed to minimize fixed end hazards to motor vehicles and if the bridge is an interchange structure, to minimize sight distance restrictions at ramp intersections.

It is recommended that bikeway bridge railings or fences placed between traffic lanes and bikeways be at least 54 inches high to minimize the likelihood of bicyclists falling over the railings. Standard bridge railings which are lower than 46 inches can be retrofitted with lightweight upper railings or chain link fence suitable to restrain bicyclists. See Index 208.10(6) for guidance regarding bicycle railing on bridges.

Separate highway overcrossing structures for bikeway traffic shall conform to Department standard pedestrian overcrossing design loading. The minimum clear width shall be the paved width of the approach bikeway but not less than 8 feet. If pedestrians are to use the structure, additional width is recommended.

(2) Surface Quality. The surface to be used by bicyclists should be smooth, free of potholes, and the pavement edge uniform. For rideability on new construction, the finished surface of bikeways should not vary more than ¼ inch from the lower edge of an 8-foot long straight edge when laid on the surface in any direction.

Table 1003.6 indicates the recommended bikeway surface tolerances for Class II and III bikeways developed on existing streets to minimize the potential for causing bicyclists to lose control of their bicycle (Note: Stricter tolerances should be achieved on new bikeway construction.) Shoulder rumble strips are not suitable as a riding surface for bicycles. See the California MUTCD, Chapter 3B for additional information regarding rumble strip design considerations for bicycles.

(3) Drainage Grates, Manhole Covers, and Driveways. Drainage inlet grates, manhole covers, etc., on bikeways should be designed and installed in a manner that provides an adequate surface for bicyclists. They should be maintained flush with the surface when resurfacing.

Table 1003.6
Bikeway Surface Tolerances

Direction of Travel	Grooves (1)	Steps (2)
Parallel to travel	No more than ½" wide	No more than ⅜" high
Perpendicular to travel	---	No more than ¾" high

Notes:

(1) Groove--A narrow slot in the surface that could catch a bicycle wheel, such as a gap between two concrete slabs.

(2) Step--A ridge in the pavement, such as that which might exist between the pavement and a concrete gutter or manhole cover; or that might exist between two pavement blankets when the top level does not extend to the edge of the roadway.

Drainage inlet grates on bikeways shall have openings narrow enough and short enough to assure bicycle tires will not drop into the grates (e.g., reticuline type), regardless of the direction of bicycle travel. Where it is not immediately feasible to replace existing grates with standard grates designed for bicycles, 1" x ¼" steel cross straps should be welded to the grates at a spacing of 6 inches to 8 inches on centers to reduce the size of the openings adequately.

Corrective actions described above are recommended on all highways where bicycle travel is permitted, whether or not bikeways are designated.

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.

(4) At-grade Railroad Crossings and Cattle Guards. Whenever it is necessary to cross railroad tracks with a bikeway, special care must be taken to assure that the safety of bicyclists is protected. The bikeway crossing should be at least as wide as the approaches of the bikeway. Wherever possible, the crossing should be straight and at right angles to the rails. For on-street bikeways where a skew is unavoidable, the shoulder (or bike lane) should be widened, if possible, to permit bicyclists to cross at right angles (see Figure 1003.6A). 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. Where hazards to bicyclist cannot be avoided, appropriate signs should be installed to warn bicyclists of the danger.

All railroad crossings are regulated by the California Public Utilities Commission (CPUC). All new bike 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.

The presence of cattle guards along any roadway where bicyclists are expected should be clearly marked with adequate advance warning.

(5) Obstruction Markings. Vertical barriers and obstructions, such as abutments, piers, and other features causing bikeway constriction, should be clearly marked to gain the attention of approaching bicyclists. This treatment should be used only where unavoidable, and is by no means a substitute for good bikeway design. See the California MUTCD, Section 9C.06.

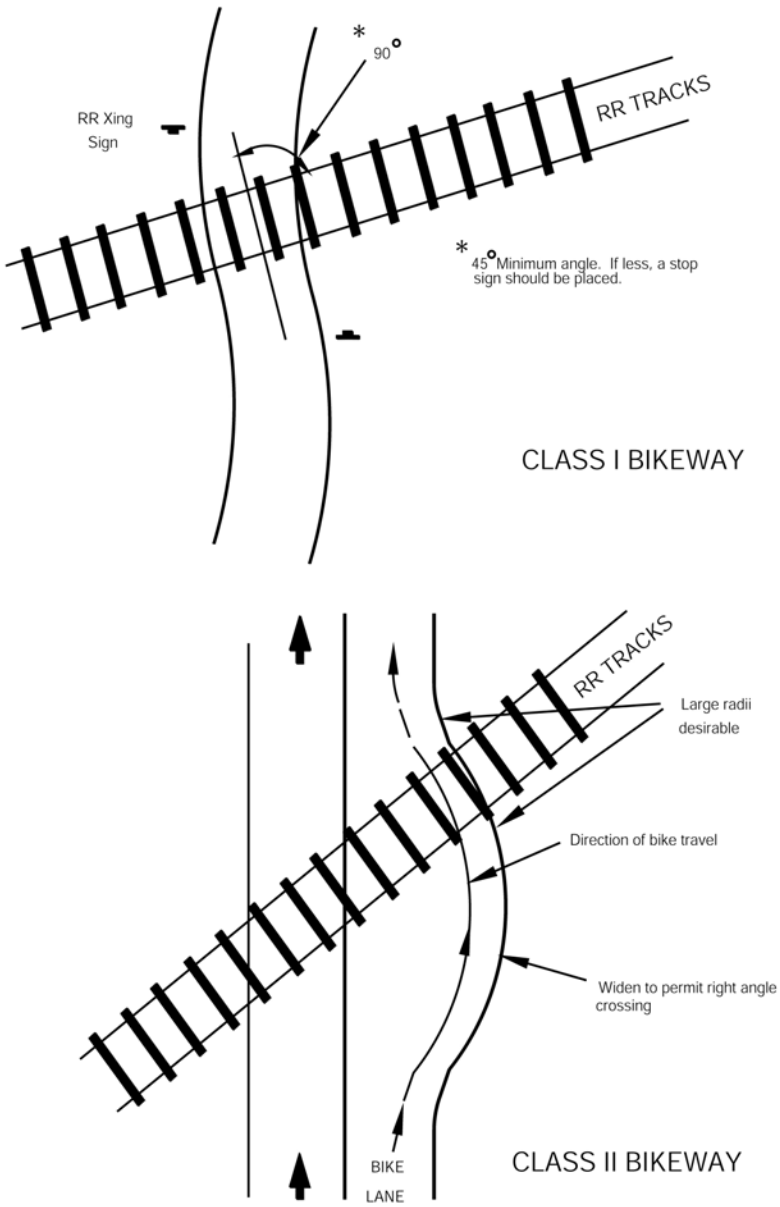



Figure 1003.6A
Railroad Crossings