CALIFORNIA
DEPARTMENT OF TRANSPORTATION

manual change transmittal

TITLE	APPROVED BY	NO.
DIVISION OF DESIGN HIGHWAY DESIGN MANUAL FIFTH & SIXTH EDITION – CHANGE 07/01/08	Helena "Lenka" Culik-Caro, Acting Chief	Date Issued: 06/27/08 Page 1 of 3
SUBJECT AREA Foreword; Table of Contents; List of Figures; List of Tables; Chapters: 60, 80, 100, 400, 600, 800; and, Index	ISSUING UNIT DIVISION OF DESIGN	
SUPERCEDES SEE BELOW FOR SPECIFIC PAGE NUMBERS	DISTRIBUTION **ALL HOLDERS OF THE 5TH & 6TH EDITION, HIGHWAY DESIGN MANUAL**	

The Foreword; Table of Contents; List of Figures; List of Tables; Chapters: 60, 80, 100, 400, 600, 800; and, the Index of the Fifth and Sixth Edition, Highway Design Manual (HDM) have been revised. The revisions and changes are described herein in the summary below with change sheets available on the Department Design website at: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm. These revisions and changes are effective July 1, 2008, and shall be applied to on-going projects in accordance with HDM Index 82.5 – Effective Date for Implementing Revisions to Design Standards.

Highway Design Manual Subscribers are encouraged to use the most recent version of the Highway Design Manual available on-line at the above website. Should a Subscriber choose to maintain a paper copy, the Subscriber 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.

A summary of the most significant revisions are as follows:

Index 62.7 — **Pavement Definitions**
The new term "Rubberized Hot Mix Asphalt (RHMA)" replaces what was formerly known as rubberized asphalt concrete (RAC).

Table 82.1A — **Mandatory Standards**
The mandatory standard (Index 614.4) that applied to pavement design life for pavement rehabilitation (CAPM) Projects was deleted as it was replaced by the recently revised CAPM Guidelines.

Table 82.1A — **Mandatory Standards**
The title of Index 645.1 was changed from Limits of Paving on Roadway Rehabilitation Projects to the more accurate subject title, Limits of Paving of Overlay Projects.

Table 82.1B — **Advisory Standards**
The advisory standards contained in Index 404.3 have been revised to match the reorganized and expanded design vehicle guidance contained in this manual change. Also a new advisory standard has been added for the 45-Foot (13.72 Meter) Bus & Motorhome Design Vehicle.

Index 107.2 — **Maintenance and Police Facilities on Freeways**
Last paragraph, last sentence – The incorrect maintenance vehicle pullout detail, Standard Plan H8, was referenced. The correct detail, Standard Plan H9, was inserted.

Index 111.1	**Material Sites and Disposal Sites, General Policy** First paragraph – Reference for further information concerning selection and procedures for disposal, staging and borrow sites was added relevant to the new Design Information Bulletin 85. Subsection (d) was revised to include the correct location where mandatory sites are to be designated for project documentation and contract purposes.
Topic 404	**Design Vehicles** Former Topics 404 and 407 have been combined, reorganized, and expanded. Design vehicle guidance has also been updated for consistency with current policies, state laws, and industry trends.
Chapter 600	**General Aspects** Minor updates in terminology and text clarifications throughout. Discussion in Index 603.3 on preventive maintenance and capital preventative maintenance (CAPM) updated to reflect updated federal definitions and new Design Information Bulletin 81 "CAPM Guidelines."
Chapter 610	**Pavement Engineering Considerations** Minor updates in terminology and text clarifications throughout. Index 612.4 "Pavement Preservation" updated to reflect updated guidance and standards on preventive maintenance and CAPM. Mandatory standard for CAPM design life removed since CAPM no longer has a defined design life (only service life extension). Mandatory standard in Index 612.5 "Rehabilitation" updated to minimum 20 year pavement design life, with minimum 40 year pavement design life required in some instances when indicated by life cycle cost analysis. Minimum and maximum design limits for lime treated subgrade documented and established in Index 614.4. Minimum strength requirement reduced from 400 psi to 300 psi. Subgrade Enhancement Geosynthetic (Index 614.5) revised to provide design value for soils with R-value less than 20 and expanded instructions for design Topic 619 "Life Cycle Cost Analysis" updated to match procedures and guidance in Life Cycle Cost Analysis Procedures Manual.
Chapter 620	**Rigid Pavement** Minor updates in terminology and text clarifications throughout. In tables in Topic 623, aggregate subbase underneath asphalt treated permeable base replaced with aggregate base. Topic 624 on pavement preservation strategies expanded and revised to include CAPM in accordance with guidance in DIB 81. Minimum designs for crack, seat, and overlay provided in Topic 625.
Chapter 630	**Flexible Pavement** Minor updates in terminology and text clarifications throughout. Topic 634 on pavement preservation strategies expanded and revised to include CAPM in accordance with guidance in DIB 81.
Chapter 640	**Composite Pavements** Minor updates in terminology and text clarifications throughout. Topic 644 on pavement preservation strategies expanded and revised to include CAPM in accordance with guidance in DIB 81.

Chapter 660	**Base and Subbase** Minor updates in terminology and text clarifications throughout. Added additional information and instructions for using treated permeable bases.
Index 802.1	**Drainage Design Responsibility, Functional Organization** Subsection (4)(f) was revised in compliance with current Federal policy as further described in Topic 805.
Index 816.6	**Time of Concentration and Travel Time** The equation for Manning's kinematic solution was converted to provide Travel Time (T_t) in minutes rather than hours for consistency with the legend in Index 816.6(1).
Table 816.6A	**Roughness Coefficients For Sheet Flow** The Table was revised to substitute the new term "hot mix asphalt", formerly known as "asphalt concrete".
Index 819.2	**Empirical Methods** The limitations on the use of the USGS Regional Flood-Frequency equations in Subsection (2), Regional Analysis Methods, following Table 819.2B, was revised to include the South Coast region.
Figure 819.2A	**Runoff Coefficients for Underdeveloped Areas** Exponents were added to the equations for the Eastern Sierras Region 5 that were mistakenly missing previously.
Figure 854.3C	**Chart for Estimating Years to Perforation of Steel Culverts** A missing exponent was added to the equation at the top of the Figure for pH of environment normally greater than 7.3 years.
Index 854.9	**Minimum Height of Cover** Instructions were added, consistent with new Culvert Low Clearance Coverage Details Guidance for situations where cover is less than shown in Table 854.9 of this manual.
Table 854.9	**Minimum Thickness of Cover for Culverts** The dimension arrow for minimum thickness of cover to top of pipe was incorrectly placed. Other minor changes were made to the figure above the table for clarification.
Index 873.3	**Armor Protection, Flexible Revetments, Streambank Protection Design** Subsection (2)(2), entitled Streambank Protection Design, fifth bullet – the following correction was applied "Not be placed on a slope steeper than 1.5H:1V" rather than "1.5V:1H".
Figure 873.3C	**Rock Slope Protection** Details were revised for Mounded Toe RSP to adjust placement limits of RSP fabric.

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

FOREWORD

Purpose

This manual was prepared by Project Delivery, Division of Design. The manual establishes uniform policies and procedures to carry out the highway design functions of the California Department of Transportation (Caltrans). 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.

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

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 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, not 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 at: 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 20 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 Fifth Edition of the Highway Design Manual is in metric units. All previous editions are now obsolete and no longer reflect current standards. All projects designed and constructed in metric units shall follow the standards in this manual per the instructions contained in Index 82.5, "Effective Date for Implementing Revisions to Design Standards". Projects designated to be designed and constructed in US Customary (English) units and standards must be designed, and constructed, in accordance with the project-related interim highway design guidance provided on the Division of Design website for projects using English units upon publishing the Sixth Edition of the HDM.

Metric Basics

Measurable Attribute - Basic Units	Unit	Expression
Length	meter	m
Mass	kilogram	kg
Luminous intensity	candela	cd
Time	second	s
Time	hour	h
Electric current	ampere	A
Thermodynamic temperature	Kelvin	K
Amount of substance	mole	mol
Volume of liquid	liter	L
Measurable Attribute - Special Names	**Unit**	**Expression**
Frequency of a periodic phenomenon	hertz	Hz (1/s)
Force	newton	N (kg·m/s^2)
Energy/work/quantity of heat	joule	J(N·m)
Power	watt	W (J/s)
Pressure/stress	pascal	Pa (N/m^2)
Celcius temperature	Celsius	°C
Quantity of electricity/electrical charge	coulomb	C
Electric potential	volt	V
Electric resistance	ohm	Ω
Luminous flux	lumen	lm
Luminance	lux	lx (lm/m^2) or (cd/m^2)
Measurable Attribute - Derived Units	**Unit**	**Expression**
Acceleration	meter per second squared	m/s^2
Area	square meter	m^2
Area	hectare	ha (10 000 m^2)
Density/mass	kilogram per cubic meter	kg/m^3
Volume	cubic meters	m^3
Velocity	meter per second	m/s
Mass	tonne	tonne (1000 kg)

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

* to be avoided where possible

Common Conversion Factors to Metric

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

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

Land Surveying Conversion Factors

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

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

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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 serviceability and structural integrity of roadway facilities on the State highway system. For further discussion, see Topic 603.

(37) *Pavement Service Life.* Is the actual period of time that a newly constructed or rehabilitated pavement structure performs satisfactorily before reaching its terminal serviceability or a condition that requires major rehabilitation or reconstruction. Because of the many independent variables involved, pavement service life may be considerably longer or shorter than the design life of the pavement. For further discussion, see Topic 612.

(38) *Pavement Structure.* See Pavement.

(39) *Pumping.* The ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab under traffic. This phenomena is especially pronounced with saturated structural sections.

(40) *Raveling.* Progressive disintegration of the surface course on asphalt concrete pavement by the dislodgement of aggregate particles and binder.

(41) *Rehabilitation.* Work undertaken to extend the service life of an existing facility. This includes placement of additional surfacing and/or other work necessary to return an existing roadway, including shoulders, to a condition of structural or functional adequacy, for the specified service life. This might include the partial or complete removal and replacement of portions of the pavement structure. Rehabilitation is divided into pavement rehabilitation activities and roadway rehabilitation activities (see Indexes 603.3 and 603.4).

(42) *Resurfacing.* A supplemental surface layer or replacement layer placed on an existing pavement to restore its riding qualities and/or to increase its structural (load carrying) strength.

(43) *Rigid Pavement.* These are pavements with a rigid surface course (typically Portland cement concrete or a variety of specialty cement mixes for rapid strength concretes) which may incorporate underlying layers of stabilized or unstabilized base or subbase materials. These types of pavements rely on the substantially higher stiffness rigid slab to distribute the traffic loads over a relatively wide area of underlying layers and the subgrade. Some rigid slabs have reinforcing steel to help resist cracking due to temperature changes and repeated loading.

(44) *Roadbed.* The roadbed is that area between the intersection of the upper surface of the roadway and the side slopes or curb lines. The roadbed rises in elevation as each increment or layer of subbase, base or surface course is placed. Where the medians are so wide as to include areas of undisturbed land, a divided highway is considered as including two separate roadbeds.

(45) *Asphalt Rubber Binder.* A blend of asphalt binder modified with crumb rubber modifier (CRM) that may include less than 15 percent CRM by mass.

(46) *Rubberized Hot Mix Asphalt (RHMA).* Formerly known as rubberized asphalt concrete (RAC). RHMA is a material produced for hot mix applications by mixing either asphalt rubber or asphalt rubber binder with graded aggregate. RHMA may be gap- (RHMA-G) or open- (RHMA-O) graded.

(47) *R-value.* See California R-Value.

(48) *Serviceability.* The ability at time of observation of a pavement to serve traffic (automobiles and trucks) which use the facility. The primary measure of serviceability is the Present Serviceability Index (PSI), which ranges from 0 (impossible road) to 5 (perfect road).

(49) *Settlement.* Localized vertical displacement of the pavement structure due to slippage or consolidation of the underlying foundation, often resulting in pavement deterioration, cracking and poor ride quality.

(50) *Structural Section.* See Pavement Structure.

(51) *Structural Section Drainage System.* See Pavement Drainage System.

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

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

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

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

62.8 Traffic

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

(2) *Delay.* The time lost while traffic is impeded by some element over which the driver has no control.

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

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

(5) *Design Volume.* A volume determined for use in design, representing traffic expected to use the highway. Unless otherwise stated, it is an hourly volume.

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

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

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

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

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

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

(12) *Speed.*

(a) Design Speed--A speed selected to establish specific minimum geometric design elements for a particular section of highway.

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

Table 82.1A
Mandatory Standards

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(1) Caltrans-only Mandatory Standard.

(2) Authority to approve deviations from this Mandatory Standard is delegated to the Chief, Office of Pavement Design.

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(1) Caltrans-only Mandatory Standard.

(2) Authority to approve deviations from this Mandatory Standard is delegated to the Chief, Office of Pavement Design.

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

(2) Authority to approve deviations from this Mandatory Standard is delegated to the Chief, Office of Pavement Design.

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(1) Caltrans-only Mandatory Standard.

(2) Authority to approve deviations from this Mandatory Standard is delegated to the Chief, Office of Pavement Design.

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504.4 Freeway-to-freeway Connector Lane Drop Tapers

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Index 612.6 Traffic Loading for Temporary Pavements and Detours

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Topic 701 **Fences**

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CHAPTER 900 LANDSCAPE ARCHITECTURE

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Topic 904 **Vista Point Design Standards and Guidelines**

Index 904.3 Road Connections to Vista Points

during the period the local road is used to provide continuity for State highway traffic. A cooperative agreement is usually required to establish terms of financing, construction, maintenance, and liability. If the local agency wants more than the minimum work needed to accommodate traffic on the local road during its use as a State highway, such betterments are to be financed by the local agency.

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

Topic 107 - Roadside Installations

107.1 Roadway Connections

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

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

107.2 Maintenance and Police Facilities on Freeways

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

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

107.3 Location of Border Inspection Stations

Other agencies require vehicles entering California to stop at buildings maintained by these agencies for inspection of vehicles and cargoes. No such building, parking area, or roadway adjacent to the parking area at these facilities should be closer than 10 m from the nearest edge of the ultimate traveled way of the highway.

Topic 108 - Coordination With Other Agencies

108.1 Divided Nonfreeway Facilities

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

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

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

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

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

108.2 Bus Loading Facilities

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

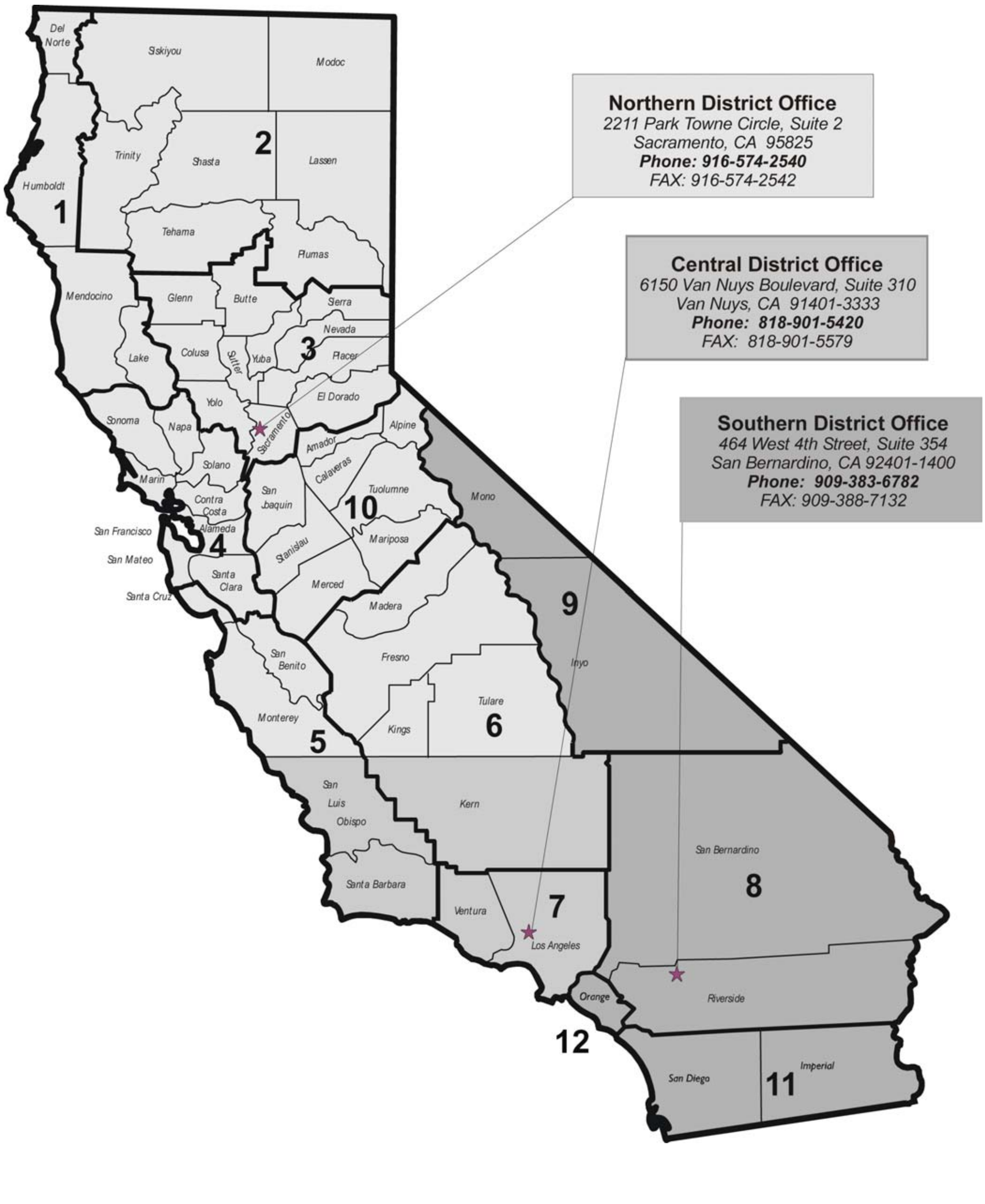
During the early phases of the design process, the District must send to the PUC, governing bodies of local jurisdictions, and common carriers or transit authorities operating in the vicinity, a map showing the proposed location and type of interchanges, with a request for their comments regarding bus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

(2) *Conferences and Hearings.* No conferences or hearings are to be held where all of the contacted agencies say that bus loading facilities are not required on the proposed freeway. The freeway should be designed without bus loading facilities in these cases.

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

Prior to the conference, the District should prepare geometric designs of the bus loading facilities for the purpose of making cost estimates and determining the feasibility of providing the facilities. Bus loading facilities must be approved by the District Director with concurrence from the Design Coordinator (see Topic 82 for approvals).

Figure 110.12
California Mining and Tunneling Districts



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

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

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

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

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

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

Topic 111 - Material Sites and Disposal Sites

111.1 General Policy

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

(a) Materials investigations and environmental studies of local materials sources should be made to the extent necessary to provide a basis for study and design. Location and capacity of available disposal sites should be determined for all projects requiring disposal of more than 7500 m^3 of clean material. Sites for disposal of any significant amount of material in sensitive areas should be considered only where there is no practical alternative.

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

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

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

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

operations, and will increase the size of the intersection and therefore crossing distances for bicyclists and pedestrians. When a right angle cannot be provided due to physical constraints, the interior angle should be designed as close to 90 degrees as is practical, but should not be less than 75 degrees. Mitigation should be considered for the affected intersection design features. (See Figure 403.3). A 75 degree angle does not unreasonably increase the crossing distance or generally decrease visibility.

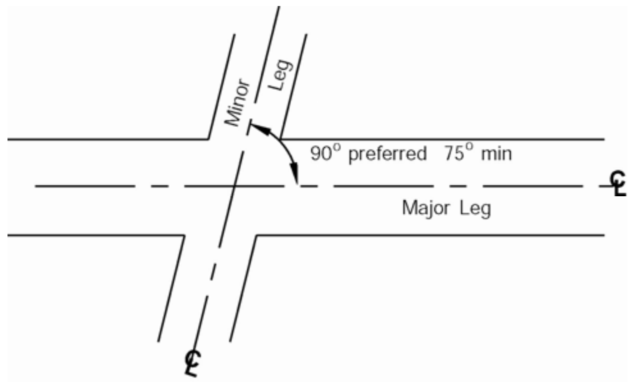
When existing intersection angles are less than 75 degrees, the following retrofit improvement strategies should be considered:

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

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

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

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



403.4 Points of Conflict

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

403.5 Speed-change Areas

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

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

403.6 Turning Traffic

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

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

403.7 Refuge Areas

The shadowing effect of traffic islands may be used to provide refuge areas for turning and crossing vehicles. Adequate shadowing provides refuge for a vehicle waiting to cross or enter an uncontrolled traffic stream. Similarly, channelization also may provide a more efficient crossing of two or more traffic streams by permitting drivers to select a time gap in one traffic stream at a time.

Traffic islands also may serve the same purposes for pedestrians and disabled persons.

403.8 Prohibited Turns

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

403.9 Effective Signal Control

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

403.10 Installation of Traffic Control Devices

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

403.11 Summary

- Give preference to the major move(s).
- Reduce areas of conflict.
- Cross traffic at right angles or skew no more than 75 degrees. (90 degrees preferred.)
- Separate points of conflict.
- Provide speed-change areas and separate turning lanes where appropriate.
- Provide adequate width to shadow turning traffic.
- Restrict undesirable moves with traffic islands.
- Coordinate channelization with effective signal control.
- Install signs in traffic islands when necessary, but avoid built-in hazards.

403.12 Precautions

- Striping is usually preferable to curbed islands, especially adjacent to high-speed traffic where curbing can be an obstruction to out-of-control vehicles.
- Where curbing must be used, first consideration should be given to mountable curbs. Barrier curbs are usually justified only where protection of pedestrians is a primary consideration.
- Avoid complex intersections that present multiple choices of movement to the driver.
- Traffic safety should be considered. Accident records provide a valuable guide to the type of channelization needed.

Topic 404 - Design Vehicles

404.1 General

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

Some terminology is vital to understanding the engineering concepts related to design vehicles.

Tracking width is the total width needed by the tires to traverse a curve; it is the distance measured along the curve radius from the outer front tire track to the inner rear tire track as the vehicle traverses around a curve. This width is used to determine the edge of pavement.

Offtracking is the difference between the paths of the front and rear wheels of a vehicle as it negotiates a turn.

Swept width is the total width needed by the vehicle body to traverse a curve; it is the distance measured along the curve radius from the outer front corner of the body path to the inner rear corner of the body as the vehicle traverses around a curve. This width is used to determine clearance.

404.2 Design Tools

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

- The turning templates in Figures 404.5A through H are a design aid for determining the swept width and/or tracking width of large vehicles as they maneuver through a turn. The templates can be used as overlays to evaluate the adequacy of the geometric layout of a curve or intersection when reproduced on clear film and scaled to match the highway drawings.
- Computer software can draw the swept width and/or tracking width along any design curve within a CADD drawing program such as MicroStation or AutoCADD. Dimensions taken from the vehicle diagrams in Figures 404.5A through H may be inputted into the computer program if the vehicle is not already included in the software library. The software can also create a vehicle turn template that conforms to any degree curve desired.

404.3 Design Vehicles and Related Definitions

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

(a) STAA Routes. STAA allows certain longer trucks called STAA Trucks to operate on the National Network. After STAA was enacted, the Department evaluated State routes for STAA truck access and created Terminal Access and Service Access routes which, together with the National Network, are called the STAA Network. Terminal Access routes allow STAA access to terminals and facilities. Service Access routes allow STAA trucks 1.6 km access off the National Network, but only at identified exits and only for designated services. Service Access routes are primarily local roads. A “Truck Network Map,” indicating the National Network routes and the Terminal Access routes is posted on the Office of Truck Services website and is also available in printed form.

(b) STAA Design Vehicle. The STAA vehicle is a truck tractor-semitrailer with the following dimensions: the maximum length of the semitrailer is 14.63 m; the kingpin-to-rear-axle (KPRA) distance is unlimited by law, although the semitrailer length usually limits this distance to about 13.11 m; the maximum body and axle width is 2.59 m; the tractor length and overall length are unlimited, (Note: a truck tractor is a non-load-carrying vehicle). The STAA Design Vehicle is shown in Figures 404.5A and B.

<u>The STAA Design Vehicle in Figures 404.5A or B should be used in the design of all projects on the National Network and on Terminal Access routes.</u> In some cases, factors such as cost, right of way, environmental issues, local agency desires and the type of community being served may limit the use of the STAA design vehicle template. In those cases, other appropriate templates should be used. This STAA design vehicle was used to

designate the existing Terminal Access and Service Access routes. The truck tractor on this vehicle has a 6.10 m wheelbase that was common in the 1980's.

(c) STAA Vehicle – Long Tractor. Since the 1980's, many truck tractors have longer wheelbases, a few reaching 7.62 m and even up to 9.14 m. The STAA Vehicle – Long Tractor in Figure 404.5C illustrates a truck tractor with a wheelbase of 7.62 m. In recent years, the highway system has experienced an increase in the number of STAA – Long Tractor vehicles. This longer STAA vehicle combination requires a wider swept width and a longer minimum radius than the current standard STAA design vehicle.

(d) STAA Vehicle – 16.15 Meter Trailer. Another category of vehicle allowed only on STAA routes has a maximum 16.15 m trailer, a maximum 12.19 m KPRA for two or more axles, a maximum 11.58 m KPRA for a single axle, and unlimited overall length. This vehicle is not to be used as the design vehicle as it is not the worst case for offtracking due to its shorter KPRA. The STAA Design Vehicle should be used instead.

(2) *California Legal.*

(a) California Legal Routes. Virtually all State routes off the STAA Network are California Legal routes. There are two types of California Legal routes; the regular California Legal routes and the KPRA Advisory Routes. Advisory routes have signs posted that state the maximum KPRA length that the route can accommodate without the vehicle offtracking outside the lane. KPRA advisories range from 9.14 m to 11.58 m, in 0.61 m increments. California Legal vehicles are allowed to use both types of California Legal routes. California Legal vehicles can also use the STAA Network. However, STAA trucks are not allowed on any California Legal routes. The "Truck Network Map" indicating the California Legal routes is posted on the Office of Truck Services website and is also available in printed form.

(b) California Legal Design Vehicle. The California Legal vehicle is a truck tractor-semitrailer with the following dimensions: the maximum overall length is 19.81 m; the maximum KPRA distance is 12.19 m for semitrailers with two or more axles, and 11.58 m for semitrailers with a single axle; the maximum width is 2.59 m. The California Legal Design Vehicle is shown in Figures 404.5D and E.

<u>The California Legal Design Vehicle in Figures 404.5D and E should be used in the design of all interchanges and intersections on California Legal routes and California Legal KPRA Advisory routes for both new construction and rehabilitation projects.</u>

(3) *12.19 Meter Buses.*

(a) 12.19 Meter Bus Routes. All single-unit vehicles, including buses and motor trucks up to 12.19 m in length, are allowed on virtually every route in California.

(b) 12.19 Meter Bus Design Vehicle. The 12.19 Meter Bus Design Vehicle shown in Figure 404.5F is an AASHTO standard. Its 7.62 m wheelbase and 12.19 m length are typical of city transit buses and some intercity buses. At intersections where truck volumes are light or where the predominate truck traffic consists of mostly 3-axle units, the 12.19 m bus may be used. Its wheel path sweeps a greater width than 3-axle delivery trucks, as well as smaller buses such as school buses.

(4) *13.72 Meter Buses & Motorhomes.*

(a) 13.72 Meter Bus & Motorhome Routes. Buses and motorhomes over 12.19 m in length, up to and including 13.72 m in length, are allowed in California on certain routes. The 13.72 m tour bus became legal on the National Network in 1991 and later allowed on some State routes in 1995. The 13.72 m motorhome became legal in 2001, but only on those routes where the 13.72 m buses were already allowed. A "Motorcoach and Motorhome Map" indicating where these longer buses and motorhomes are allowed and where they are not allowed is posted on the Office of Truck Services website and is also available in printed form. (Note: Motorcoach is a common industry term for tour bus).

(b) 13.72 Meter Bus & Motorhome Design Vehicle. The 13.72 Meter Bus & Motorhome Design Vehicle shown in Figure 404.5G is used by the Caltrans Truck Size Unit for the longest allowable buses and motorhomes. Its wheelbase is 8.69 m. It is also similar to the AASHTO standard 13.72 m bus.

<u>The 13.72 Meter Bus & Motorhome Design Vehicle shown in Figure 404.5G should be used in the design of all interchanges and intersections on all green routes on the "Motorcoach and Motorhome Map" for both new construction and rehabilitation projects.</u> Check also the larger standard design vehicles on these routes as required – the STAA Design Vehicle and the California Legal Design Vehicle in Indexes 404.4(1) and (2).

(5) 18.29 Meter Articulated Buses.

(a) 18.29 Meter Articulated Bus Routes. The articulated bus is allowed a length of up to 8.29 m per CVC 35400(b)(3)(A). This bus is used primarily by local transit agencies for public transportation. There is no master listing of such routes. Local transit agencies should be contacted to determine possible routes within the proposed project.

(b) 18.29 Meter Articulated Bus Design Vehicle. The 18.29 Meter Articulated Bus Design Vehicle shown in Figure 404.5H is an AASHTO standard. The routes served by these buses should be designed to accommodate the 18.29 Meter Articulated Bus Design Vehicle.

404.4 Design Considerations

Both the tracking width and swept width should be considered in the design of left and right turns where use of the roadway by design vehicles is warranted.

Tracking width lines delineate the path of the vehicle tires as the vehicle moves through the turn. Tracking width lines should not encroach onto adjacent or opposing lanes. Tracking width lines may encroach onto paved shoulders.

For projects where the tracking width lines are shown to encroach onto paved shoulders, the shoulder pavement structure must be engineered to sustain the weight of the design vehicle. If curb and gutter are present and any portion of the gutter pan is likewise encroached, the gutter pan must be engineered to match the adjacent shoulder pavement structure. See Topic 613 for general traffic loading considerations, and Index 626.2(4) for tied rigid shoulder guidance.

In addition, swept width lines delineate the path of the vehicle body as the vehicle moves through the turn and will therefore always exceed the tracking width. Swept width lines should not encroach onto adjacent or opposing lanes. Swept width lines may encroach onto paved shoulders, and may encroach beyond the edge of pavement. However, swept width lines may not encroach upon obstacles including but not limited to curbs, islands, sign structures, traffic delineators/channelizers, traffic signals, lighting poles, guardrails, trees, cut slopes, and rock outcrops. Swept width lines do not include side mirrors or other appurtenances allowed by the California Vehicle Code, thus, accommodation to non-motorized users of the facility should be considered.

If both the tracking width and swept width lines meet the design guidance stated above, then the geometry is adequate for that design vehicle. If either the tracking width or swept width lines do not meet the design guidance stated above, then an alternative design should be used, such as roadway widening. However, before roadway widening is proposed, consideration should be given to pedestrian crossing distance, motor vehicle speeds, truck volumes, alignment, bicycle lane width, sight distance, and the presence of on-street parking.

Tracking width and swept width may also be used when determining adequate widths and clearances, for example, when designing tight curves on narrow mountainous roads, tight intersections with obstructions, and construction zones. Swept width is useful for determining corner radii, positioning island noses, establishing clearance to bridge piers, placing signal poles and other hardware at intersections, and determining the width of a channelized turn lane.

Note that both the STAA Design Vehicle and the California Legal Design Vehicle have a template with 15.24 m (minimum) and 18.29 m (longer) radii. The STAA – Long Tractor has a template with an 18.29 m radius, which is the minimum radius for this vehicle.

The longer radius templates are more conservative and are preferred. The longer radius templates develop less swept width and leave a margin of error for the truck driver. The longer radius templates should be used for conditions where the vehicle may not be required to stop before entering the intersection.

The minimum radius template can be used if the longer radius template does not clear all obstacles. The minimum radius templates demonstrate the tightest turn that the vehicles can navigate, assuming a speed of less than 16 km/h.

Also note that there are three templates for buses and motorhomes: (1) the 12.19 m bus, (2) the 13.72 m bus and motorhome, and (3) the 18.29 m articulated bus. Each radius is the minimum that the bus or motorhome can navigate, assuming a speed of less than 16 km/h.

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

404.5 Turning Templates & Vehicle Diagrams

Figures 404.5A through H are computer-generated turning templates at an approximate scale of 1:500 and their associated vehicle diagrams for the design vehicles described in Index 404.3. The radius of the template is measured to the outside front wheel path at the beginning of the curve. Figures 404.5A through H contain the terms defined as follows:

(1) Tractor Width – Width of tractor body.

(2) Trailer Width – Width of trailer body.

(3) Tractor Track – Tractor axle width, measured from outside face of tires.

(4) Trailer Track – Trailer axle width, measured from outside face of tires.

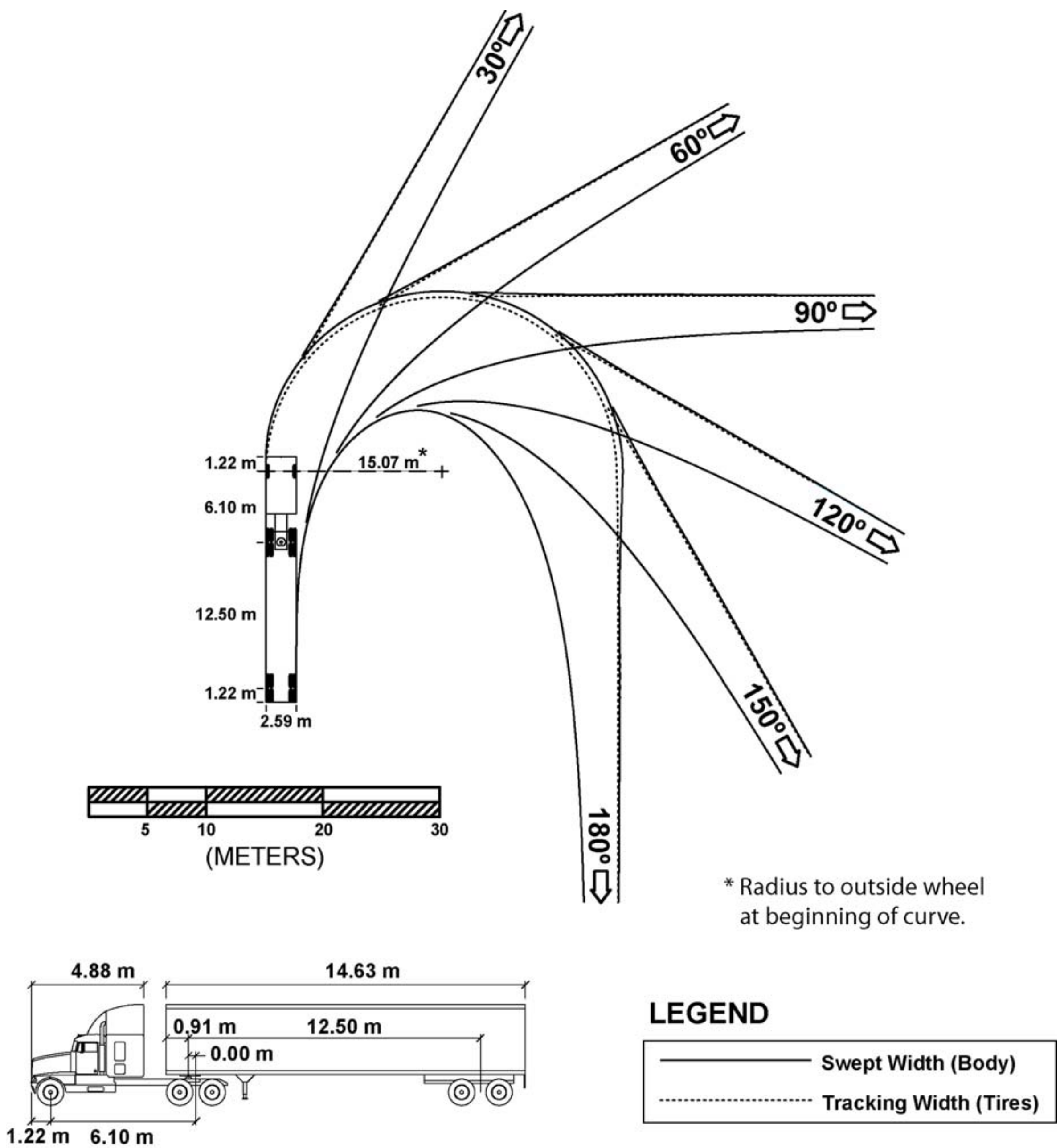
(5) Lock To Lock Time - The time in seconds that an average driver would take under normal driving conditions to turn the steering wheel of a vehicle from the lock position on one side to the lock position on the other side. The *AutoTurn* default is 6 seconds.

(6) Steering Lock Angle - The maximum angle that the steering wheels can be turned. It is further defined as the average of the maximum angles made by the left and right steering wheels with the longitudinal axis of the vehicle.

(7) Articulating Angle - The maximum angle between the tractor and semitrailer.

Figure 404.5A

STAA Design Vehicle
15 Meter Radius



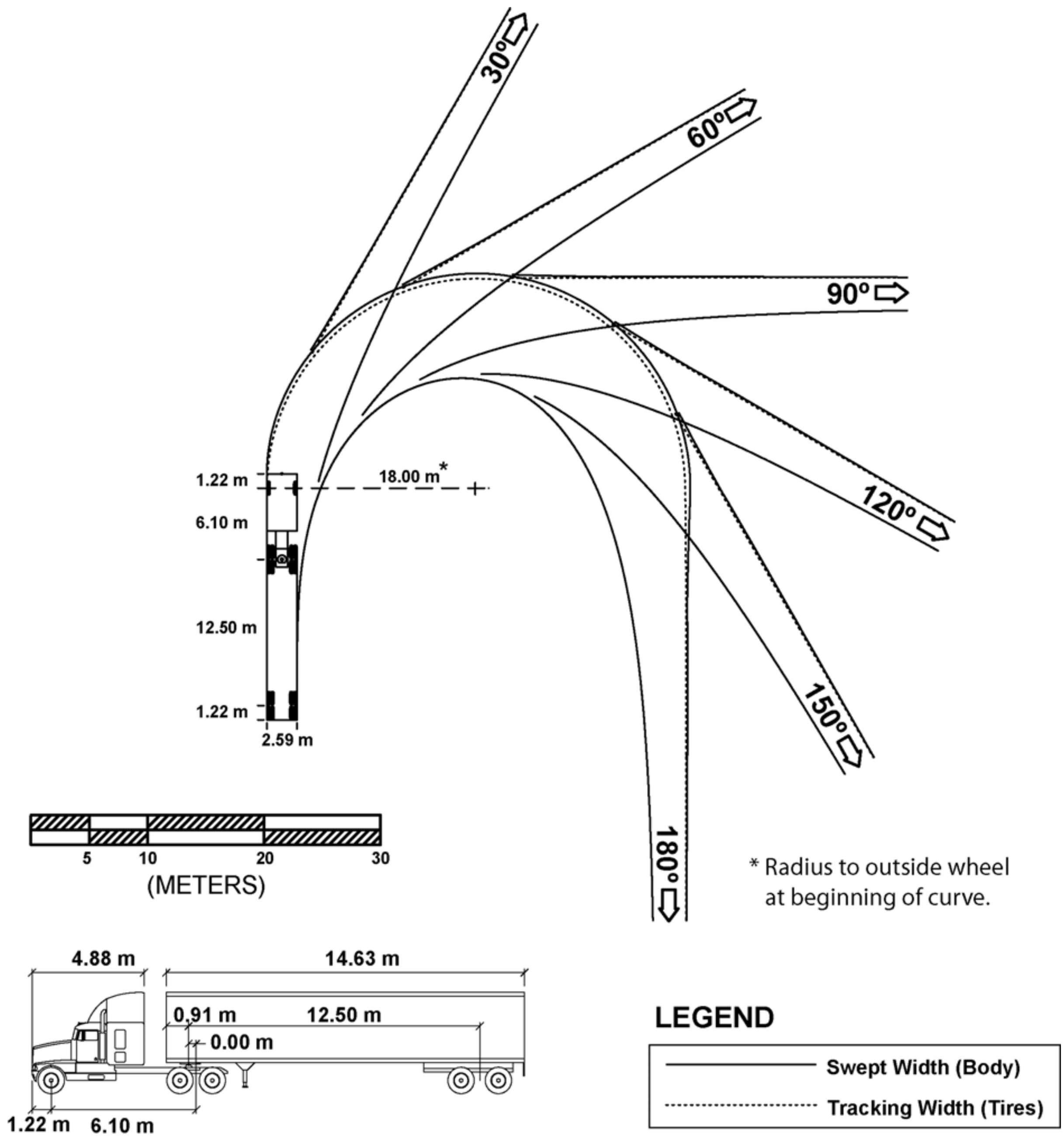
STAA - STANDARD

Tractor Width	: 2.59 m	Lock to Lock Time	: 6 seconds
Trailer Width	: 2.59 m	Steering Lock Angle	: 26.3 degrees
Tractor Track	: 2.59 m	Articulating Angle	: 70 degrees
Trailer Track	: 2.59 m		

Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5B

STAA Design Vehicle
18 Meter Radius



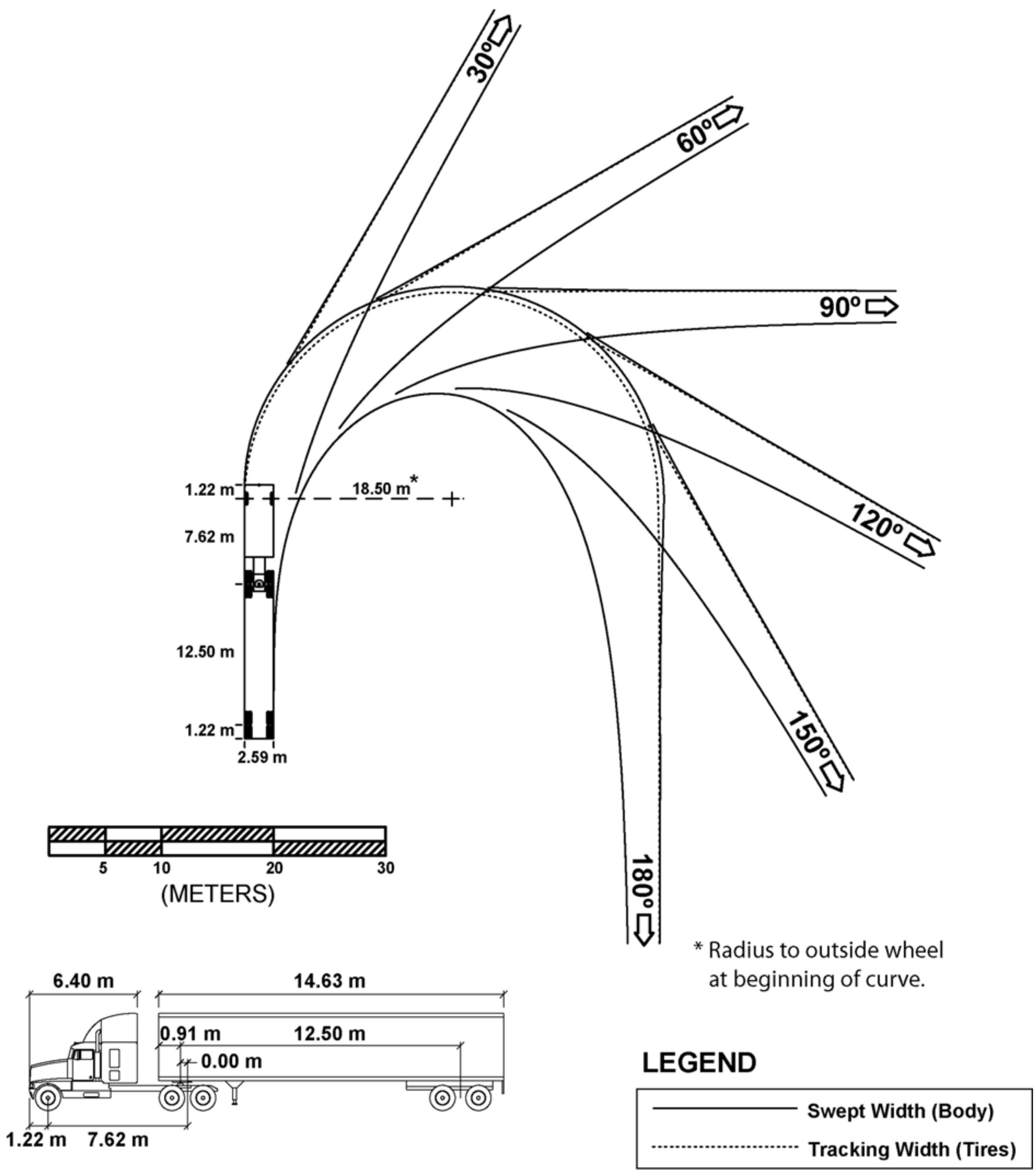
STAA - STANDARD

Tractor Width	: 2.59 m	Lock to Lock Time	: 6 seconds
Trailer Width	: 2.59 m	Steering Lock Angle	: 26.3 degrees
Tractor Track	: 2.59 m	Articulating Angle	: 70 degrees
Trailer Track	: 2.59 m		

Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5C

STAA – Long Tractor



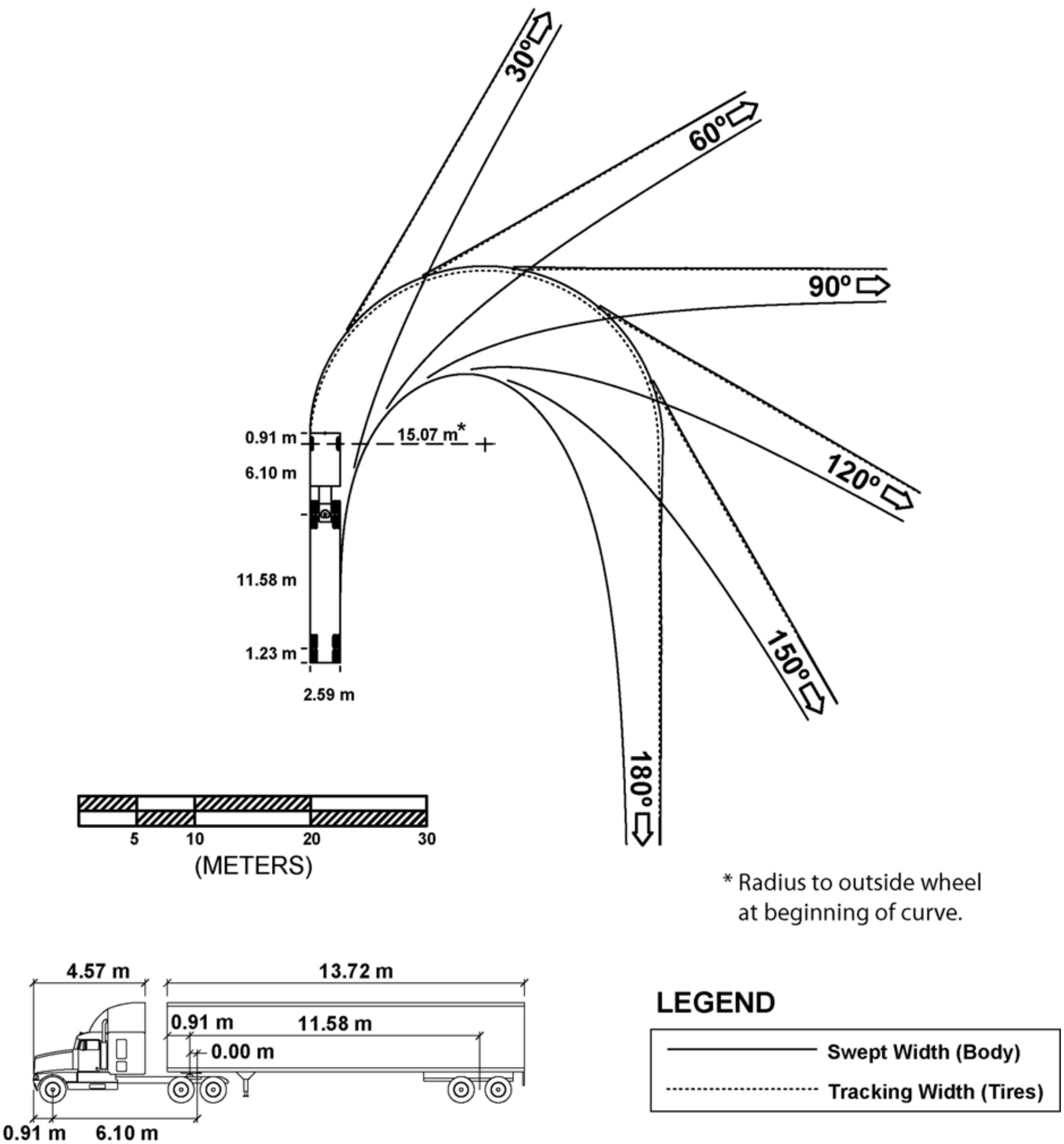
STAA -- LONG TRACTOR

Tractor Width	: 2.59 m	Lock to Lock Time	: 6 seconds
Trailer Width	: 2.59 m	Steering Lock Angle	: 26.3 degrees
Tractor Track	: 2.59 m	Articulating Angle	: 70 degrees
Trailer Track	: 2.59 m		

Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5D

California Legal Design Vehicle
15 Meter Radius



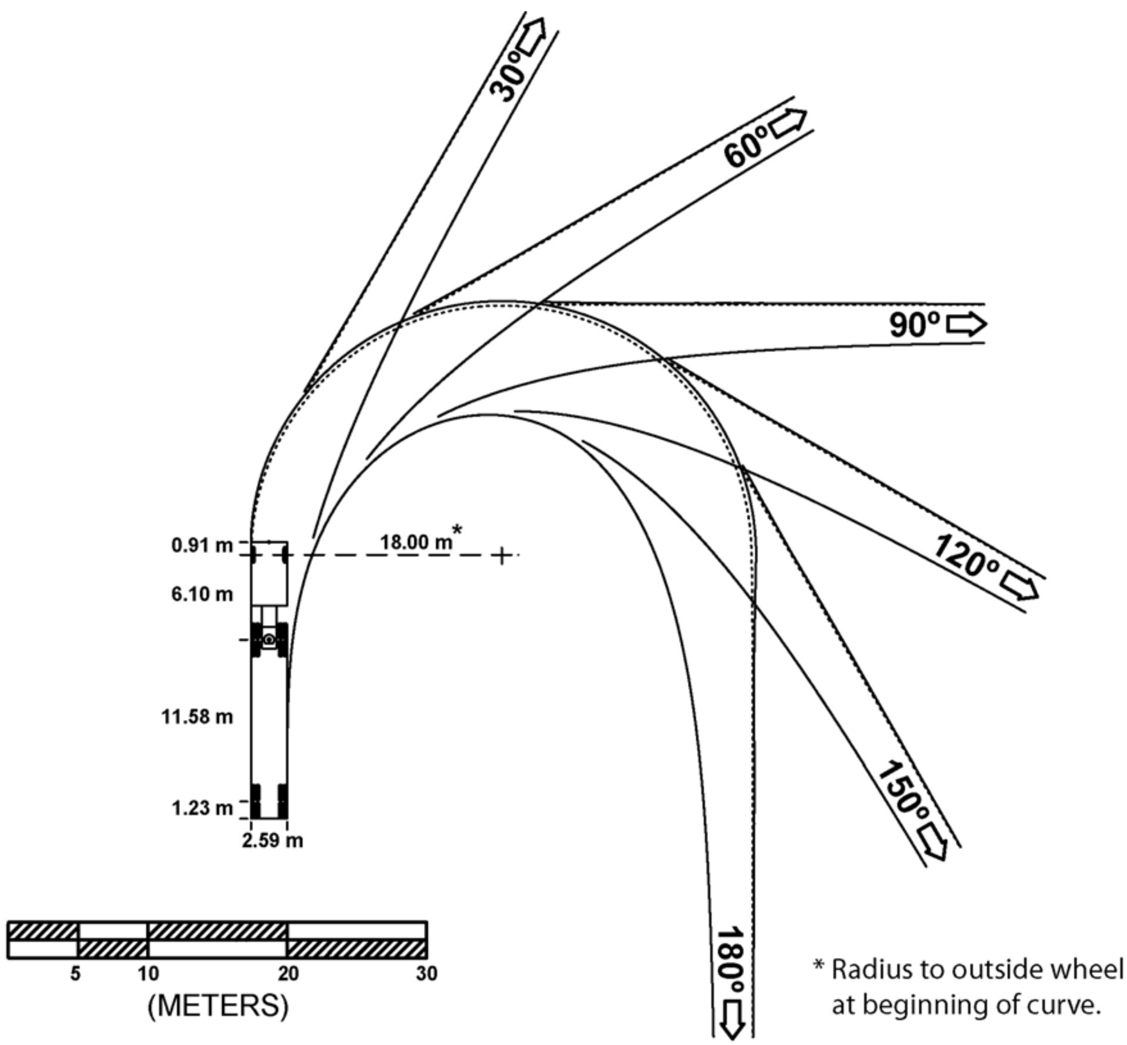
CA LEGAL - 19.81 Meter

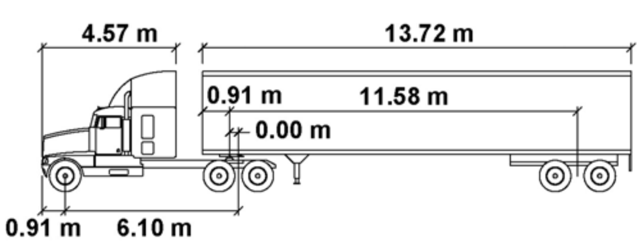
Tractor Width	: 2.59 m	Lock to Lock Time	: 6 seconds
Trailer Width	: 2.59 m	Steering Lock Angle	: 26.3 degrees
Tractor Track	: 2.59 m	Articulating Angle	: 70 degrees
Trailer Track	: 2.59 m		

Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5E

California Legal Design Vehicle
18 Meter Radius





LEGEND

————	**Swept Width (Body)**
··········	**Tracking Width (Tires)**

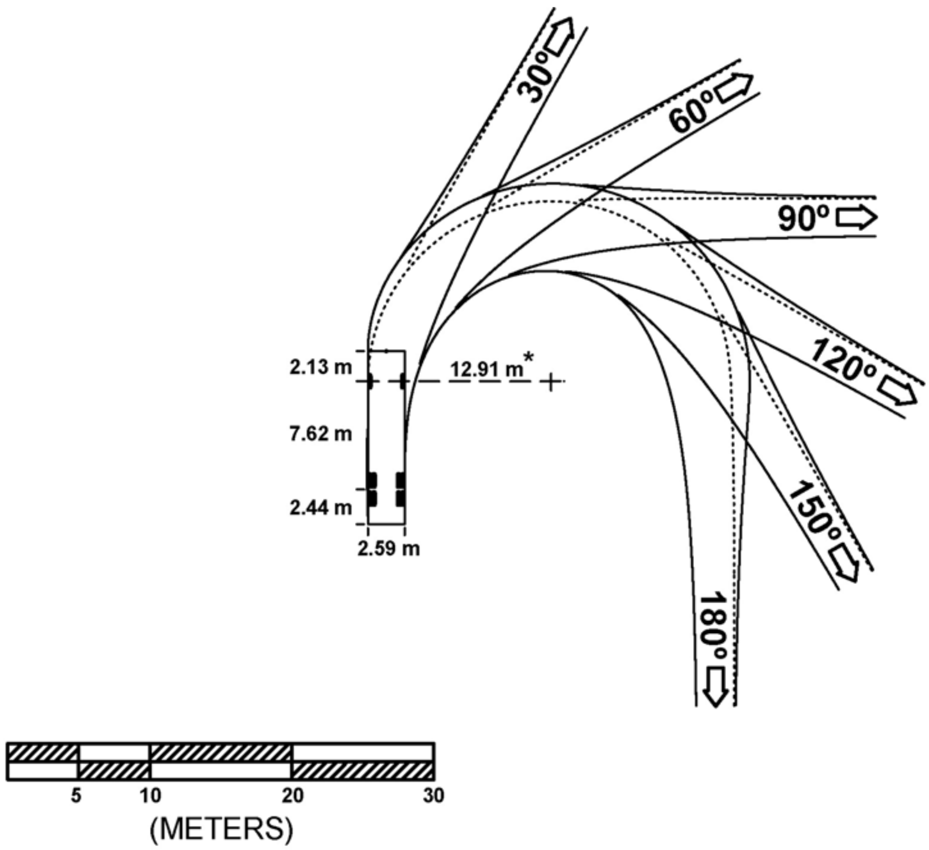
CA LEGAL - 19.81 Meter

Tractor Width	**: 2.59 m**	**Lock to Lock Time**	**: 6 seconds**
Trailer Width	**: 2.59 m**	**Steering Lock Angle**	**: 26.3 degrees**
Tractor Track	**: 2.59 m**	**Articulating Angle**	**: 70 degrees**
Trailer Track	**: 2.59 m**		

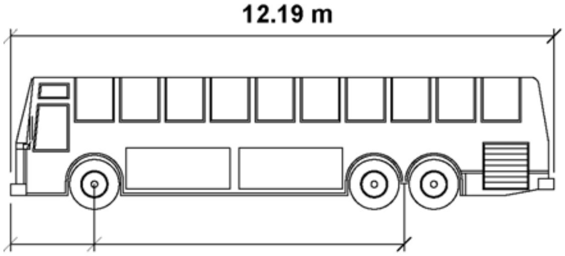
Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5F

12.19 Meter Bus Design Vehicle



* Radius to outside wheel
at beginning of curve.



LEGEND

————	**Swept Width (Body)**
- - - - - -	**Tracking Width (Tires)**

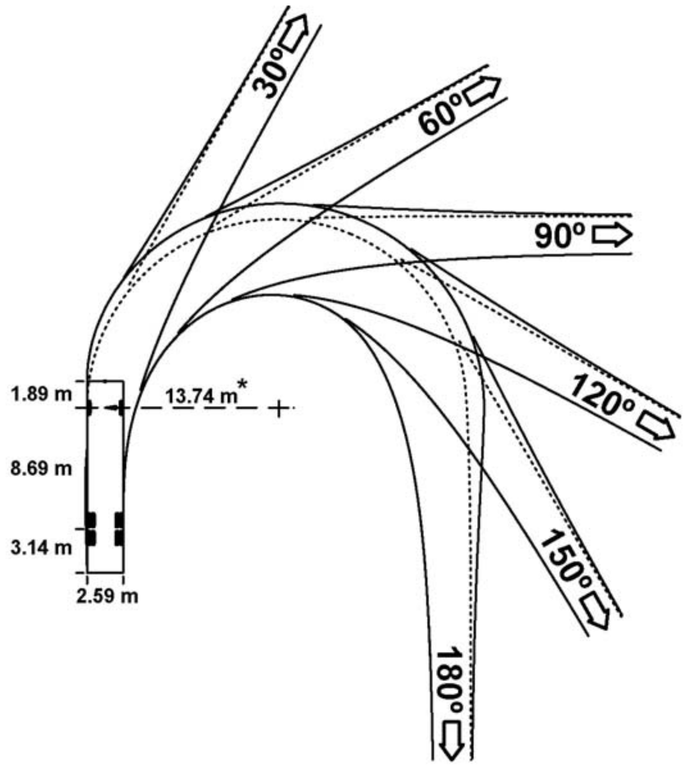
12.19 Meter BUS

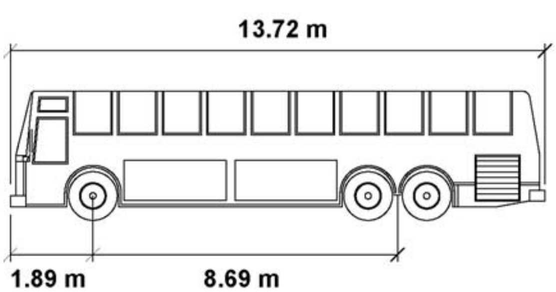
Width	**: 2.59 m**
Track	**: 2.59 m**
Lock to Lock Time	**: 6 seconds**
Steering Lock Angle	**: 41 degrees**

Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5G

13.72 Meter Bus & Motorhome Design Vehicle





* Radius to outside wheel at beginning of curve.

LEGEND

————————	**Swept Width (Body)**
-------------------	**Tracking Width (Tires)**

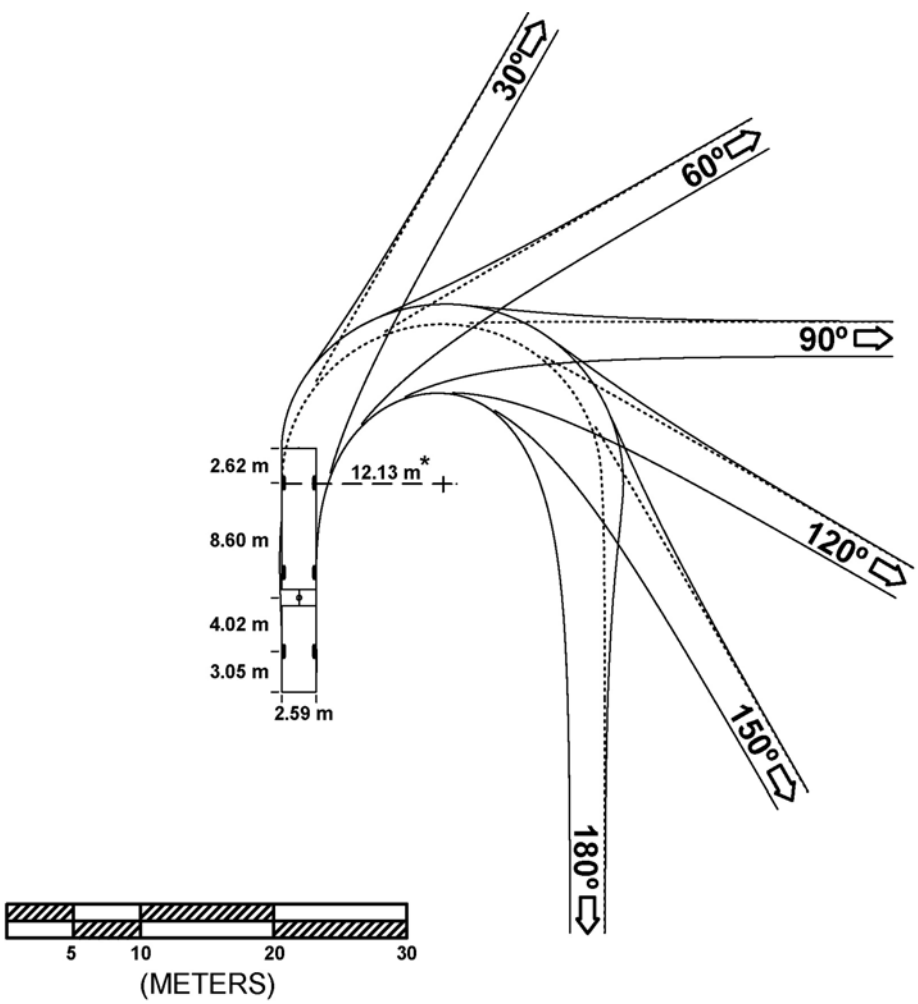
13.72 Meter BUS

Width : 2.59 m
Track : 2.59 m
Lock to Lock Time : 6 seconds
Steering Lock Angle: 44.3 degrees

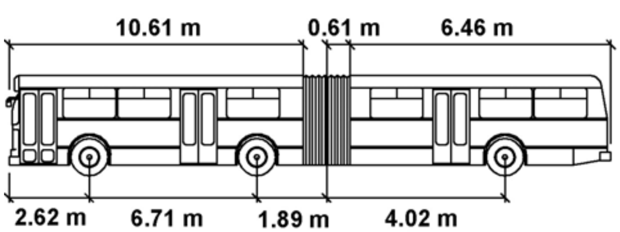
Note: For definitions, see Indexes 404.1 and 404.5.

Figure 404.5H

18.29 Meter Articulated Bus Design Vehicle



* Radius to outside wheel
at beginning of curve.



LEGEND

————————	**Swept Width (Body)**
------------------	**Tracking Width (Tires)**

18.29 Meter ARTICULATED BUS

Width : 2.59 m
Track : 2.59 m
Lock to Lock Time : 6 seconds
Steering Lock Angle : 38.3 degrees
Articulating Angle : 50 degrees

Note: For definitions, see Indexes 404.1 and 404.5.

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% and longer than 2 km (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 3 m plus the shoulder width of the major road but not less than 4 m.** Corner sight distance is to be measured from a 1070 mm height at the location of the driver on the minor road to a 1300 mm object height in the center of the approaching lane of the major road. If the major road has a median barrier, a 600 mm 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 unmitigable 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 1070 mm eye height and the 150 mm 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 the AASHTO publication, "A Policy on Geometric Design of Highways and Streets", the Headquarters Traffic Liaison and the Project Development Coordinator.

Table 405.1A
Corner Sight Distance
(7-1/2 Second Criteria)

Design Speed (km/h)	Corner Sight Distance (m)
40	90
50	110
60	130
70	150
80	170
90	190
100	210
110	230

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 Intersections & Route Direction Changes, with or without Signals	X	X	X

(1) Using stopping sight distance between an eye height of 1070 mm and an object height of 1300 mm. 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 3.6 m.** Under certain circumstances (listed below), left-turn lane widths of 3.3 m or as narrow as 3.0 m 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 3.3 m.
- 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 3.0 m. When double left-turn lanes are warranted under these same circumstances the width of each lane shall be no less than 3.3 m. 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.

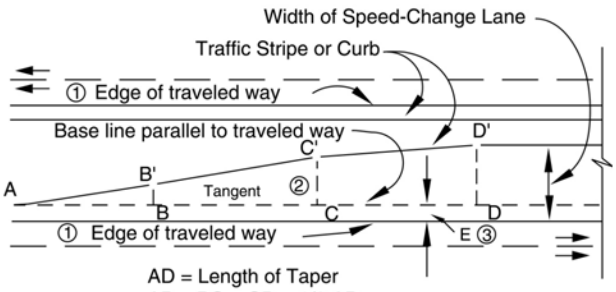
Figure 405.2A shows a standard left-turn channelization design in which all widening is to the right of approaching traffic and the deceleration lane (see below) begins at the end of the approach taper. This design should be used in all situations where space is available, usually in rural and semi-rural 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 18 m and 27 m are normally used. Where space is restricted and speeds are low, a 18 m bay taper is appropriate. On rural high-speed highways, a 36 m 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 15 to 30 km/h 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



AD = Length of Taper
AB = BC = CD = 1/3 AD
AB' & C'D' are Parabolic Curves

	LENGTH OF TAPER - meters			OFFSET DISTANCE			
	18	27	36	DD' = 3.0 m	DD' = 3.3 m	DD' = 3.6 m	
	Distance From Point "A"						
	-	-	-	0	0	0	
	1.5	2.25	3.0	0.048	0.051	0.057	
	3.0	4.5	6.0	0.186	0.207	0.225	
	4.5	6.75	9.0	0.423	0.465	0.507	
B'	6	9	12	0.75	0.825	0.90	B'
	9	13.5	18	1.50	1.65	1.80	
C'	12	18	24	2.25	2.475	2.70	C'
	13.5	20.25	27	2.58	2.84	3.10	
	15	22.5	30	2.81	3.09	3.38	
	16.5	24.75	33	2.95	3.25	3.54	
	18	27	36	3.0	3.3	3.6	

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 0.6 m along edge of traveled way for curbed medians; Use "E" = 0 m for striped medians.

Table 405.2B
Deceleration Lane Length

Design Speed (km/h)	Length to Stop (m)
50	75
60	94
70	113
80	132
90	150
100	169

(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 7.5 m per car. If the peak hour truck traffic is 10 % 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 1 m, but not more than 10 m, 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) apply 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 3.6 m (see Index 301.1). The preferred width is 4.2 m. Wider TWLTL's are occasionally provided to conform with local agency standards. However, TWLTL's wider than 4.2 m are not recommended, and in no case should the width of a TWLTL exceed 4.8 m. Additional width may encourage drivers in opposite directions to use the TWLTL simultaneously.

405.3 Right-turn Channelization

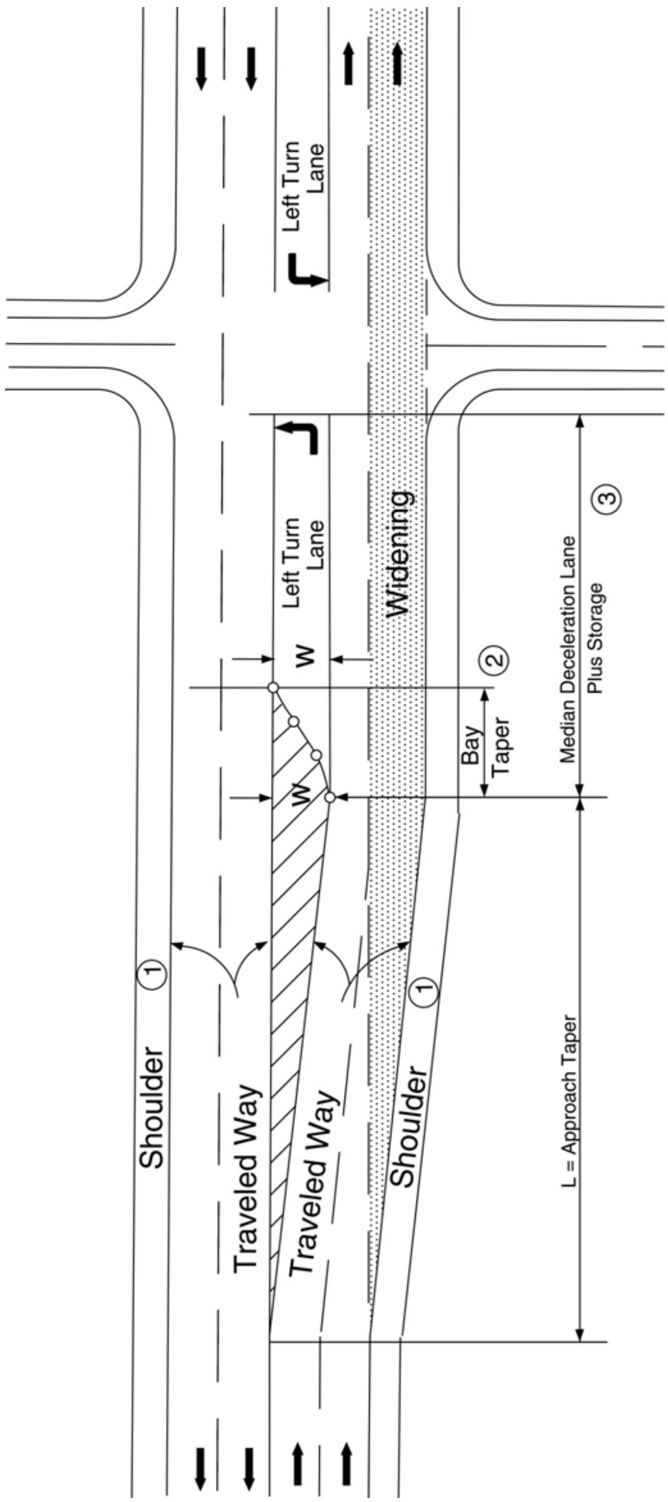
(1) *General.* For right-turning traffic, delays are less critical and conflicts less severe than for left-turning traffic. Nevertheless, right-turn lanes can be justified on the basis of capacity, analysis, and accident experience.

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 $(2/3)WV$, for $V \geq 70$ km/h ④
Or $WV^2/150$, for $V < 70$ km/h

Where L = Length of Approach Taper - meters
V = Design Speed - km/h
W = Width of Median Lane - meters

NOTES:

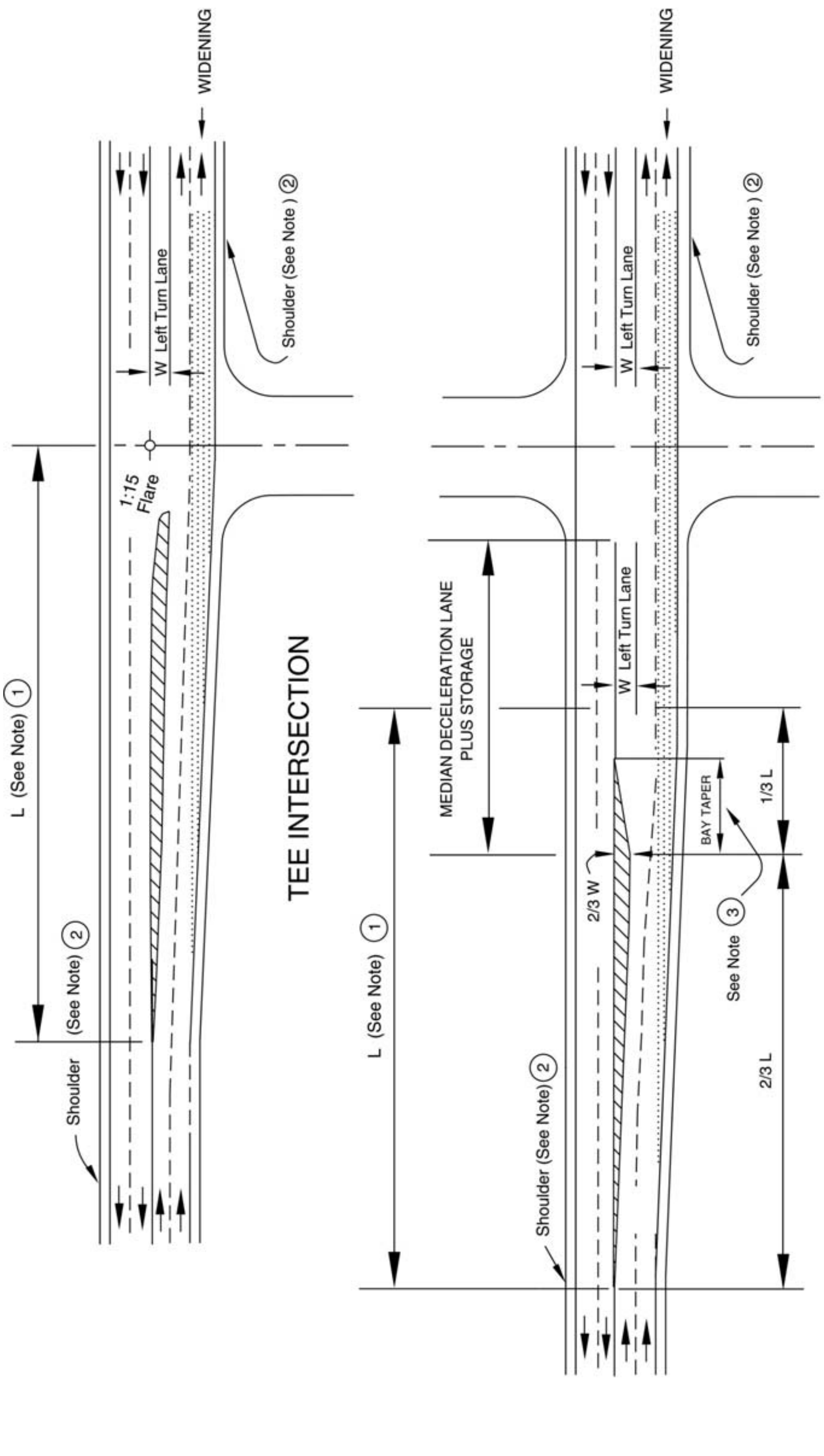
① Where width is restricted, shoulder width may be reduced and parking restricted with an approved design exception pursuant to Index 82.2.
For bicycle use, a minimum 1.2 m shoulder is required (1.5 m if gutter is present).

② Bay taper length = 18 m to 36 m. (See Table 405.2A)

③ For deceleration lane length see Table 405.2B.

④ Where both sides of roadway are widened, use a fraction of "W" that is proportional to widening on each side.

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



EQUATION

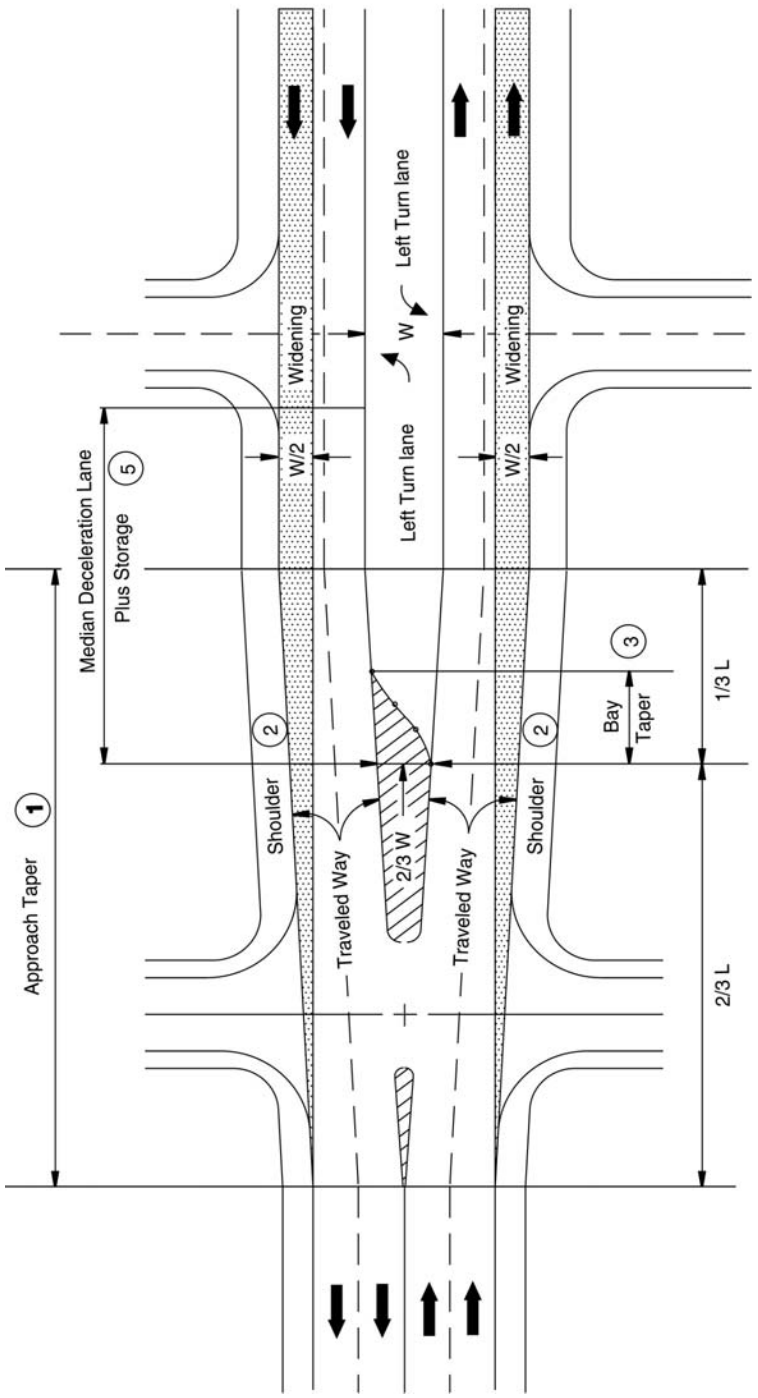
$L =$ Use $(2/3)WV$, for $V \geq 70$km/h
Or $WV^2/150$, for $V < 70$km/h

Where
L = Length of Transition - meters
W = Width of Median Lane - meters
V = Design Speed - km/h

NOTES:

1. L = 150 m Maximum
2. Where width is restricted, shoulder width may be reduced and parking restricted with an approved design exception pursuant to Index 82.2. For bicycle use, a minimum 1.2 m shoulder is required (1.5 m if gutter is present).
3. Bay taper length 18 m to 36 m (See Table 405.2A).

Figure 405.2C
Minimum Median Left-turn Channelization
(Widening on Both Sides in Urban Areas with Short Blocks)



NOTES:

1. L = 150 m Maximum
2. Where width is restricted, shoulder width may be reduced and parking restricted with an approved design exception pursuant to Index 82.2. For bicycle use, a minimum 1.2 m shoulder is required (1.5 m if gutter is present).
3. Bay taper length = 18 m to 36 m. (See Table 405.2A)
4. Assumes equal widening each side. Where widening is unequal, use a fraction that is proportional to widening on each side.
5. For deceleration lane length see Table 405.2B.

EQUATION: (4)

L = Use $(1/3)WV$, for $V \geq 70$ km/h
Or $WV^2/300$, for $V < 70$ km/h

Where L = Length of Approach Taper - Meters
W = Width of Median Lane - Meters
V = Design Speed - km/h

(2) Design Elements.

(a) Lane and Shoulder Width--**The basic lane width for right turn lanes shall be 3.6 m. Shoulder width shall be a minimum of 1.2 m.** Whenever possible, consideration should be given to increasing the shoulder width to 2.4 m 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 3.3 m 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 3.0 m with approval of a design exception.
- Shoulder widths may also be considered for reduction under constricted situations. Whenever possible, at least a 0.6 m 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 3.3 m 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 15 to 30 km/h for a lower entry speed. For example, if the main line speed is 80 km/h and a 20 km/h deceleration is permitted on the through lanes, the deceleration length may be that required for 60 km/h.

(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(2) 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.

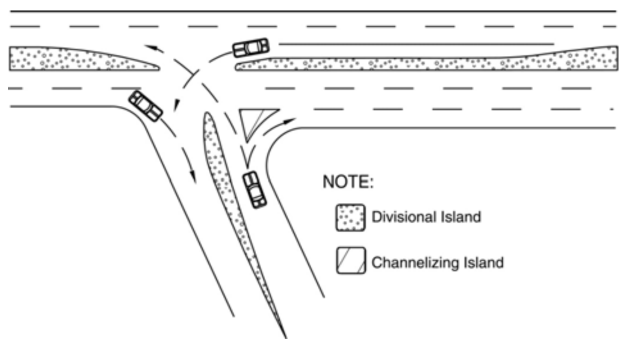
Traffic islands are also used to discourage or prohibit undesirable movements.

(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 5 m^2 in area, preferably 7 m^2. Curbed, elongated divisional islands should not be less than 1.2 m wide and 6 m long.

The approach end of each island should be offset 1 m to the left and 1.5 m 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 0.6 m left and 2.4 m 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 7.5 m 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 1 m 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.

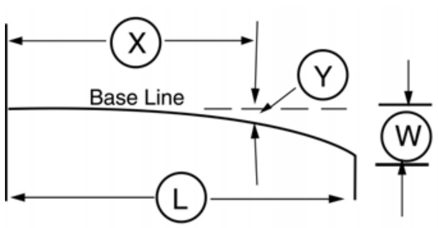
(c) Unpaved areas (small unpaved areas should be avoided).

On facilities with speeds over 75 km/h, 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.

Table 405.4

Parabolic Curb Flares Commonly Used



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

L = Length of flare in meters

W = Maximum offset in meters

X = Distance along base line in meters

Y = Offset from base line in meters

W is shown in table thus

OFFSET IN METERS FOR GIVEN "X" DISTANCE																			
Distance X / L Length of Flare	2	4	5	8	10	12	15	16	18	20	22	24	26	28	30	32	34	36	45
1:5 FLARES																			
5	0.16	0.64	1.00																
10	0.08	0.32	0.50	1.28	2.00														
15	0.05	0.21	0.33	0.85	1.33	1.92	3.00												
1:10 FLARES																			
10	0.04	1.06	0.25	0.64	1.00														
20	0.02	0.08	0.13	0.32	0.50	0.72	1.13	1.28	1.62	2.00									
30	0.01	0.05	0.08	0.21	0.33	0.48	0.75	0.85	1.08	1.33	1.61	1.92	2.25	2.61	3.00				
1:15 FLARES																			
15	0.02	0.07	0.11	0.28	0.44	0.64	1.00												
30	0.01	0.04	0.06	0.14	0.22	0.32	0.50	0.57	0.72	0.89	1.08	1.28	1.50	1.74	2.00				
45	0.01	0.02	0.04	0.09	0.15	0.21	0.33	0.38	0.48	0.59	0.72	0.85	1.00	1.16	1.33	1.52	1.71	1.92	3.00

In urban areas, speeds less than 75 km/h allow more frequent use of curbed islands. Local agency requirements and matching existing conditions are factors to consider.

405.5 Median Openings

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

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

(2) *Spacing and Location.* By a combination of interchange ramps and emergency passageways, provisions for access to the opposite side of the freeway may be provided for law enforcement, emergency, and maintenance vehicles to avoid extreme out-of-direction travel. Access should not be more frequent than at 5 km intervals. See Chapter 7 of the Traffic Manual for additional information on the design of emergency passageways. <u>Emergency passageways should be located where decision sight distance is available (see Table 201.7)</u>.

Median openings at close intervals on other types of highways create interference with fast through traffic. <u>Median openings should be spaced at intervals no closer than 500 m. If a median opening falls within 100 m of an access opening, it should be placed opposite the access opening.</u>

(3) *Length of Median Opening.* For any three or four-leg intersection on a divided highway, the length of the median opening should be at least as great as the width of the crossroads pavement, median width, and shoulders. An important factor in designing median openings is the path of the design vehicle making a minimum left turn at 8 to 15 km/h. The length of median opening varies with width of median and angle of intersecting road.

Usually a median opening of 18 m is adequate for 90 degree intersections with median widths of 6.6 m or greater. When the median width is less than 6.6 m, a median opening of 21 m is needed. When the intersection angle is other than 90 degrees, the length of median opening should be established by using truck turn templates (see Index 404.3).

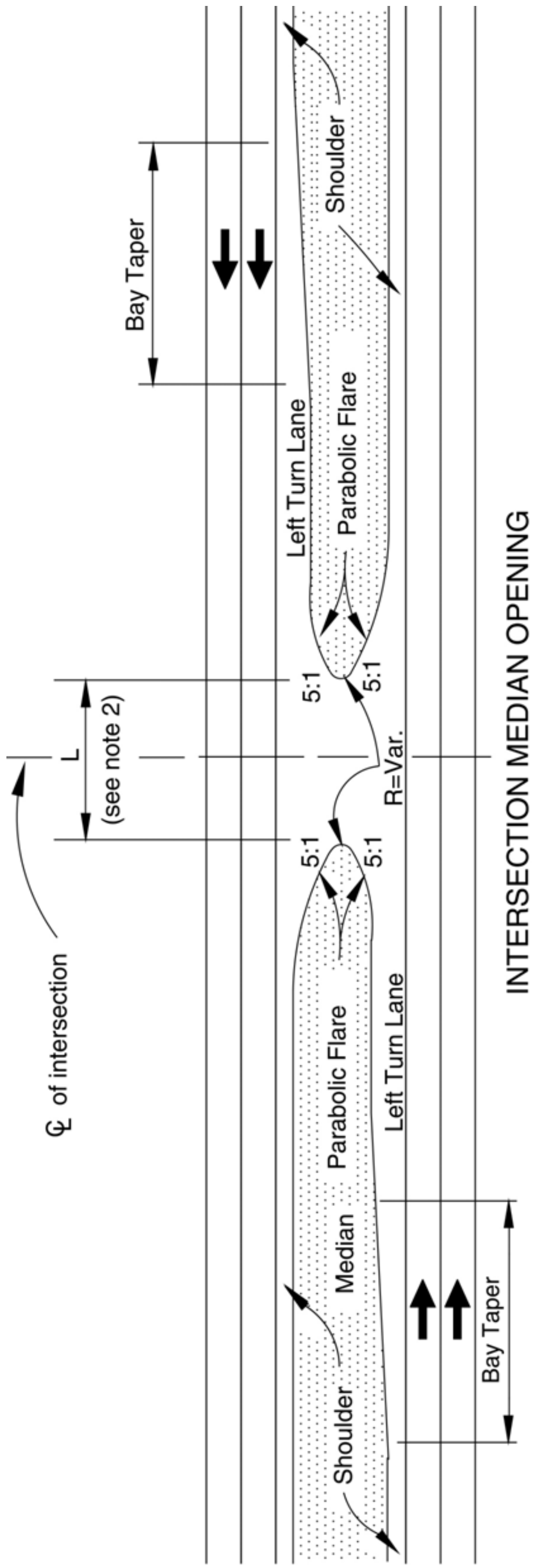
(4) *Cross Slope.* The cross slope in the median opening should be limited to 5 percent. Crossovers on curves with super elevation exceeding 5 percent should be avoided. This cross slope may be exceeded when an existing 2-lane roadbed is converted to a 4-lane divided highway. The elevation of the new construction should be based on the 5 percent cross slope requirement when the existing roadbed is raised to its ultimate elevation.

(5) *References.* For information related to the design of intersections and median openings, "A Policy on Geometric Design of Highways and Streets," AASHTO, should be consulted.

405.6 Access Control

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

Figure 405.5
Typical Design for Median Openings



NOTES:

1 - For length of bay taper, see Table 405.2A.

2 - L = Length of median opening: varies with width of median and angle of intersecting road. Usually for 90 intersection, L=18 m for median of 6.6 m and wider. L=21 m for median narrower than 6.6 m

3 - See Index 405.2.

405.7 Public Road Intersections

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

(1) Right-turn Onto the Main Highway. The combination of a circular curve joined by a 2:1 taper on the crossroads and a 22.5 m taper on the main highway is designed to fit the wheel paths of the appropriate turning template chosen by the designer.

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

(2) Right-turn Off the Main Highway. The combination of a circular curve joined by a 45 m taper on the main highway and a 4:1 taper on the crossroads is designed to fit the wheel paths of the appropriate turning template and to move the rear of the vehicle off the main highway. Deceleration and storage lanes may be provided when necessary (see Index 405.3).

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

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

405.8 City Street Returns and Corner Radii

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

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

Smaller radii of 5 to 8 m are appropriate at minor cross streets where few trucks are turning. Local agency standards may be appropriate in urban and suburban areas.

Encroachment into opposing traffic lanes should be avoided.

405.9 Widening of 2-lane Roads at Signalized Intersections

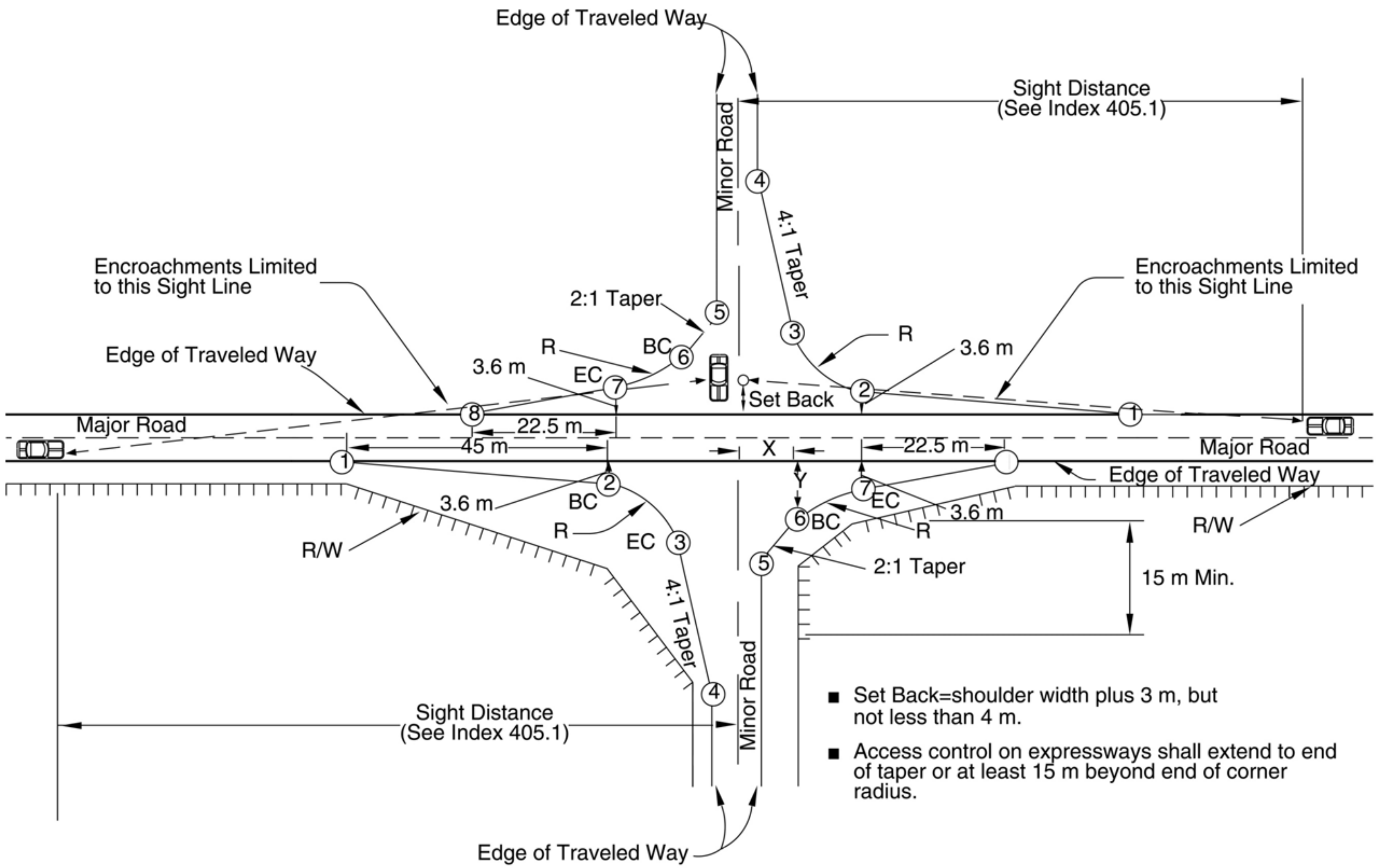
Two-lane State highways may be widened at intersections to 4-lanes whenever signals are installed. Sometimes it may be necessary to widen the intersecting road. The minimum design is shown in Figure 405.9. More elaborate treatment may be warranted by the volume and pattern of traffic movements. Unusual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

Topic 406 - Ramp Intersection Capacity Analysis

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

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

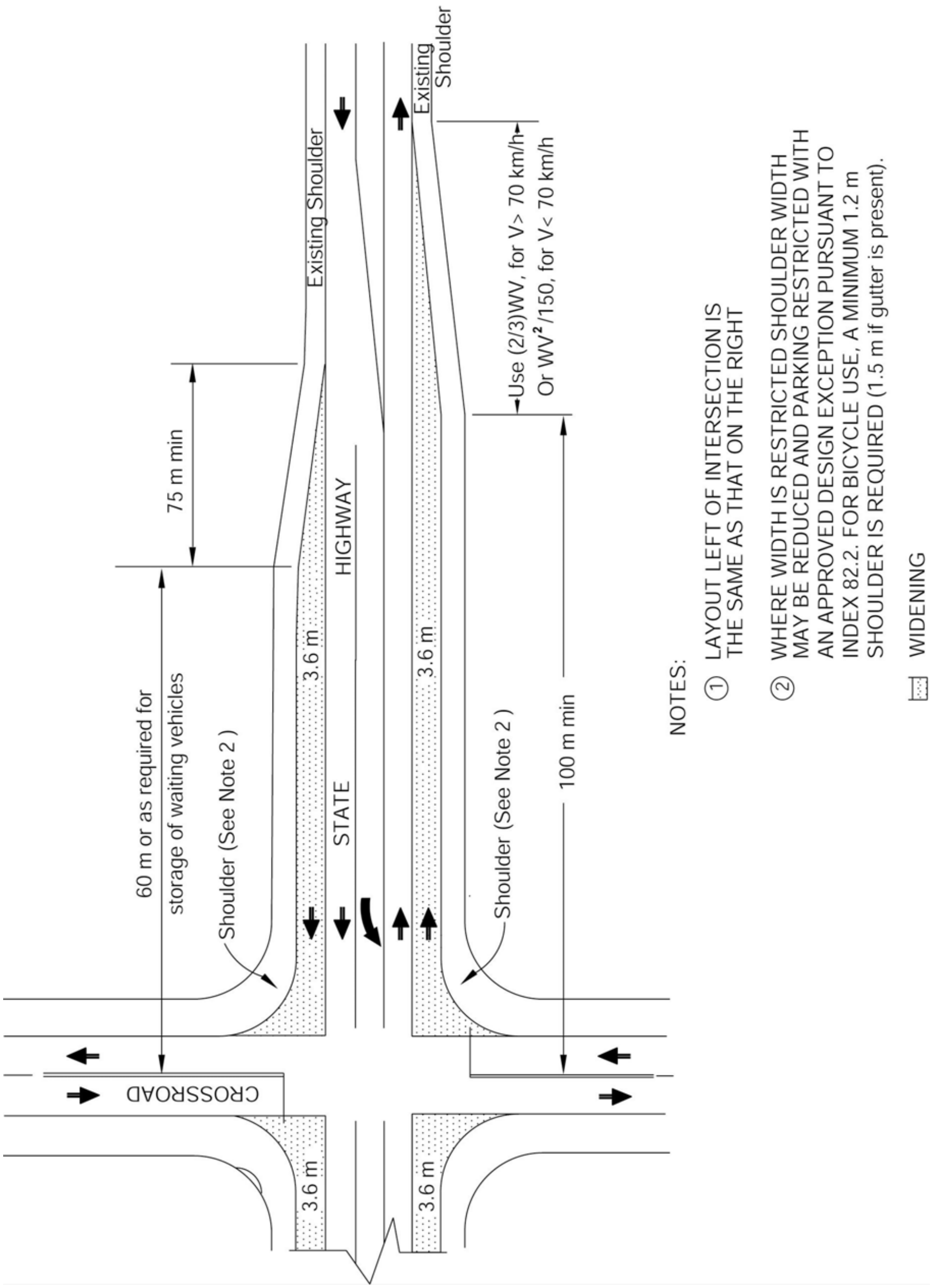
Figure 405.7
Public Road Intersections



X - Distance measured from centerline of minor road along major road - m.
Y - Offset distance measured from edge of traveled way of major road to any given point - m.

Radius of Curve	Design Vehicle	Pt ①		Pt ②		Pt ③		Pt ④		Pt ⑤		Pt ⑥		Pt ⑦		Pt ⑧	
		X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
9 m	Bus	61.26	0.0	16.26	3.6	8.25	10.39	3.6	28.97	3.6	12.20	5.47	8.46	12.10	3.6	34.60	0.0
12 m	California	64.52	0.0	19.52	3.6	8.84	12.65	3.6	33.61	3.6	16.01	6.56	10.08	15.40	3.6	37.90	0.0
15 m	STAA	67.85	0.0	22.83	3.6	9.47	14.91	3.6	38.39	3.6	22.69	9.09	11.70	20.14	3.6	42.64	0.0

Figure 405.9
Widening of Two-lane Roads at Signalized Intersections



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

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

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

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

Table 406

Traffic Flow Conditions at Intersections at Various Levels of Operation

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

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

Figure 406A
Spread Diamond

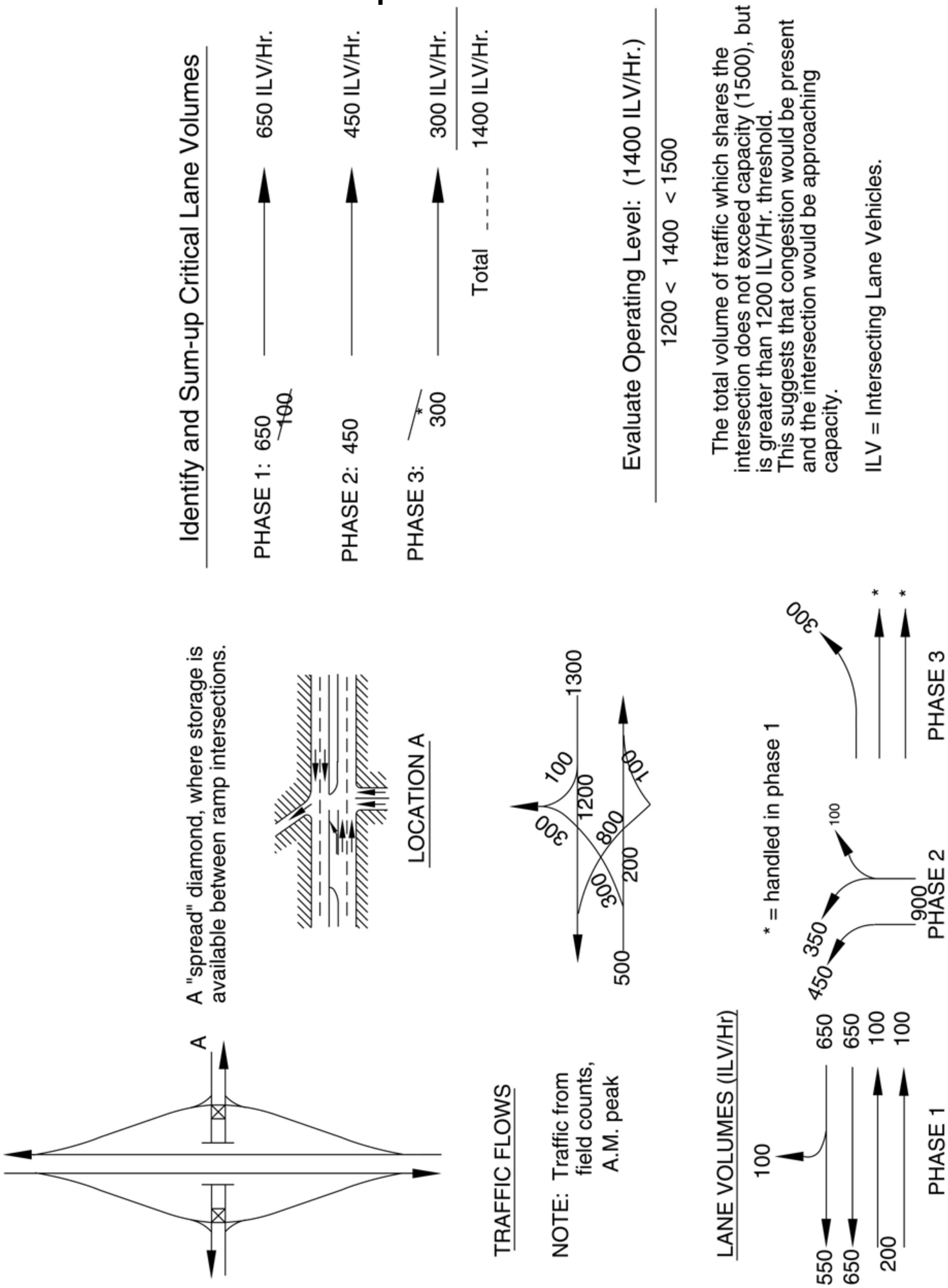
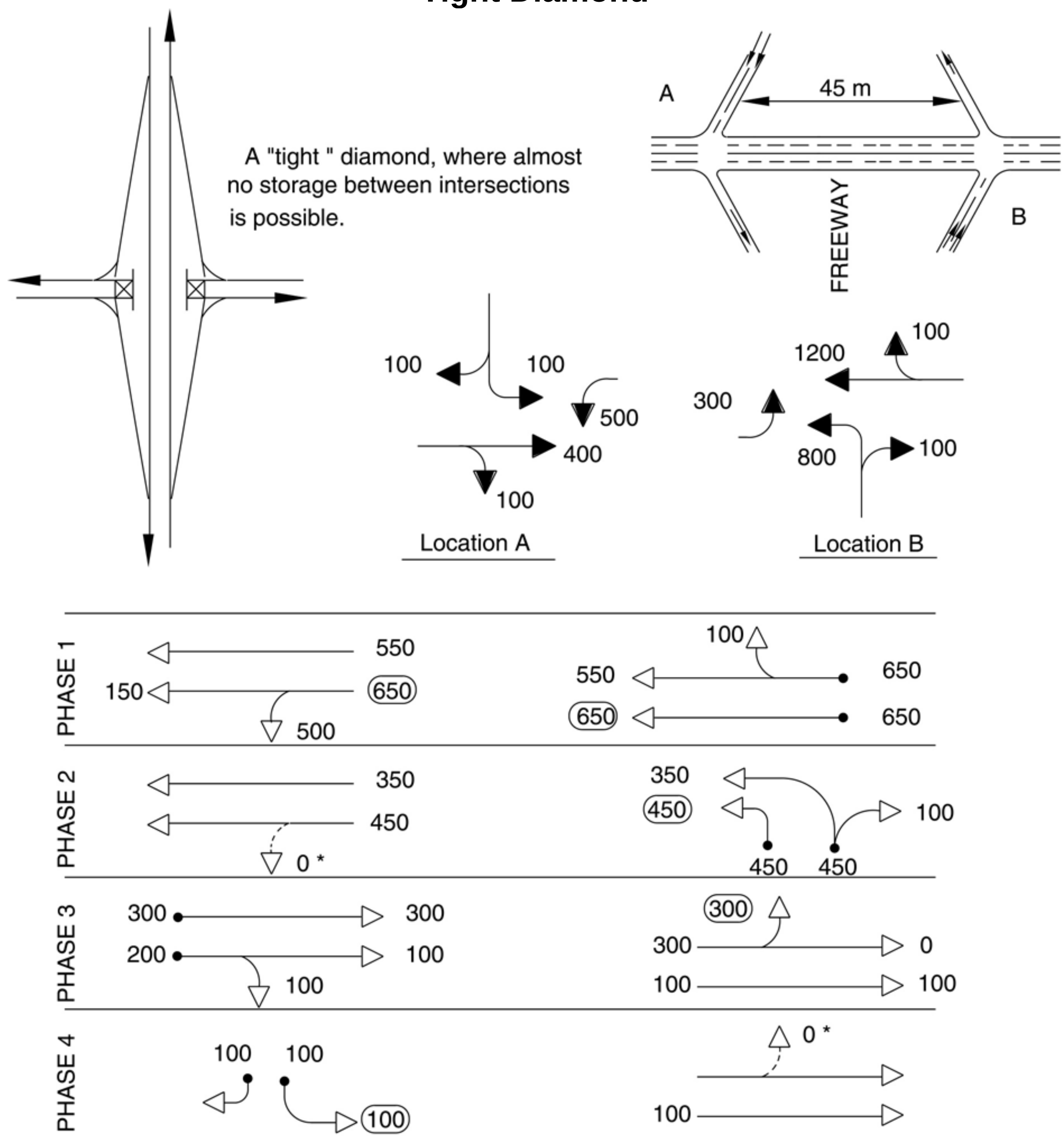


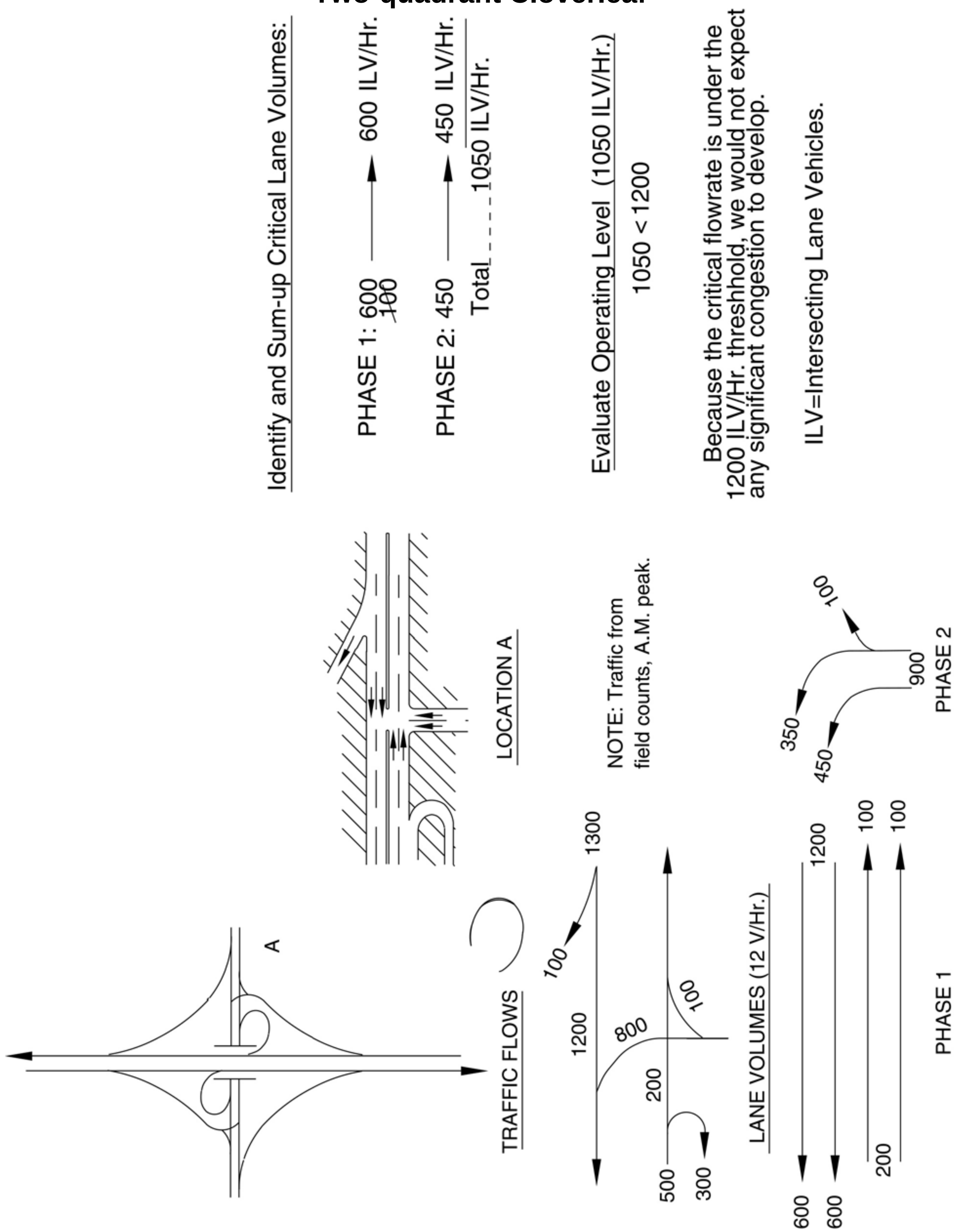
Figure 406B
Tight Diamond



*NOTE: When no storage at all is permitted, left-turn movement is cleared during this phase.

Critical Lane Volumes:	650
	450
	300
ILV=Intersecting Lane Vehicles.	100
	1500 ILV/Hr.

Figure 406C
Two-quadrant Cloverleaf



CHAPTERS 600 – 670 PAVEMENT ENGINEERING

CHAPTER 600 PAVEMENT ENGINEERING

Topic 601 - Introduction

Pavement engineering involves the determination of the type and thickness of pavement surface course, base, and subbase layers that in combination are cost effective and structurally adequate for the projected traffic loading and specific project conditions. This combination of roadbed materials placed in layers above the subgrade (also known as basement soil) is referred to as the "pavement" or the "pavement structure".

The Department guidelines and standards for pavements described in this manual are based on extensive engineering research and field experience, including the following:

- Theoretical concepts in pavement engineering and analysis.
- Data obtained from test track studies and experimental sections.
- Research on materials characteristics, testing methods, and equipment.
- Observation of performance throughout the State and the nation.

The pavement should be engineered using the standards and guidance described in this manual to ensure consistency throughout the State and provide a pavement structure that will have adequate strength, ride quality, and durability to carry the projected traffic loads for the design life of each project. The final pavement structure for each project should be based on a thorough investigation of specific project conditions including subgrade soils and structural materials, environmental conditions, projected traffic, cost effectiveness, and the performance of other pavements in the same area or similar climatic and traffic conditions. These factors are discussed in Chapter 610 of this manual.

The guidelines and standards found in this manual should be considered minimum standards and should not preclude sound engineering judgment based on experience and knowledge of the local conditions. Sound engineering judgment must still be used to determine if more stringent standards are required.

Topic 602 – Pavement Structure Layers

Index 602.1 Description

Pavement structures are comprised of one or more layers of select materials placed above the subgrade. The basic pavement layers of the roadway are shown in Figure 602.1 and discussed below.

(1) *Subgrade.* Also referred to as basement soil, the subgrade is that portion of the roadbed consisting of native or treated soil on which surface course, base, subbase, or a layer of any other material is placed. Subgrade may be composed of either in-place material that is exposed from excavation, or embankment material that is placed to elevate the roadway above the surrounding ground. Subgrade soil characteristics are discussed in Topic 614.

(2) *Subbase.* Unbound or treated aggregate/granular material that is placed on the subgrade as a foundation or working platform for the base. It functions primarily as structural support but it can also minimize the intrusion of fines from the subgrade into the pavement structure, improve drainage, and minimize frost action damage. The subbase generally consists of lower quality materials than the base but better than the subgrade soils. Subbase may not be needed in areas with higher quality subgrade (California R-value > 40) or where it is more cost effective to build a thicker base layer. Further discussion on subbase materials and concepts can be found in Chapter 660.

(3) *Base.* Select, processed, and/or treated aggregate material that is placed immediately below the surface course. It provides additional load distribution and contributes to drainage and frost resistance. Base may be one or multiple layers treated with cement, asphalt or other binder material, or may consist of untreated aggregate. In some cases, the base may include a drainage layer to drain water that seeps into the base. The aggregate in base is typically a higher quality material than that used in subbase. Further discussion on base materials and concepts can be found in Chapter 660.

(4) *Surface Course.* One or more layers of the pavement structure engineered to accommodate and distribute traffic loads, provide skid resistance, minimize disintegrating effects of climate, reduce tire/pavement noise, improve surface drainage, and minimize infiltration of surface water into the underlying base, subbase and subgrade. Sometimes referred to as the surface layer, the surface course may be composed of a single layer, constructed in one or more lifts of the same material, or multiple layers of different materials.

Depending on the type of base or subbase layers, surface courses are used to characterize pavements into the following three categories:

(a) Flexible Pavements. These are pavements engineered to bend or flex when loaded. Flexible pavements transmit and distribute traffic loads to the underlying layers. The highest quality layer is the surface course, which typically consists of one or more layers of asphalt binder mixes and may or may not incorporate underlying layers of base and/or subbase. These types of pavements are called "flexible" because the total pavement structure bends (or flexes) to accommodate deflection bending under traffic loads. Procedures for flexible pavements can be found in Chapter 630.

(b) Rigid Pavements. These are pavements with a rigid surface course typically a slab of Portland cement concrete (or a variety of specialty hydraulic cement concrete mixes used for rapid strength concrete) over underlying layers of stabilized or unstabilized base or subbase materials. These types of pavements rely on the substantially higher stiffness of the concrete slab to distribute the traffic loads over a relatively wide area of underlying layers and the subgrade. Some rigid concrete slabs have reinforcing steel to help resist cracking due to temperature changes and repeated loading. Procedures for rigid pavements can be found in Chapter 620.

(c) Composite Pavements. These are pavements comprised of both flexible (asphalt binder mixes) and rigid (cement concrete) layers over underlying layers of stabilized or unstabilized base or subbase materials. Currently, for purposes of the procedures in this manual, only pavements with a flexible layer over a rigid surface layer are considered to be composite pavements. In California, such pavements consist mostly of existing rigid pavements (typically Portland cement concrete) that have had a flexible surface course overlay such as hot mix asphalt (HMA) (formerly known as asphalt concrete), open graded friction course (OGFC) (formerly known as open graded asphalt concrete), or rubberized hot mix asphalt (RHMA) (formerly known as rubberized asphalt concrete). See Chapter 640 for additional information on composite pavements.

(5) *Non-Structural Wearing Course.* On some pavements, a non-structural wearing course is placed to protect the surface course from wear and tear from tire/pavement interaction, the weather, and other environmental factors. Examples of non-structural wearing courses include OGFC, various types of surface seals, and added surface course thickness to allow for chain wear or grinding. Although non-structural wearing courses are not given a structural value in the procedures and tables found in this manual, they will improve the service life of the pavement by protecting it from traffic and environmental effects.

(6) *Others.* Depending on the type of pavement built and the subgrade or existing soil conditions encountered, additional layers may be included in the pavement. Some of these layers include:

(a) Interlayers can be used between pavement layers or within pavement layers to reinforce pavement and/or improve resistance to reflective cracking of the pavement structure.

(b) Bond Breakers are used to prevent bonding between two pavement layers such as rigid pavement surface course to a stabilized base.

(c) Tack Coats are used to bond a layer of asphalt binder mix to underlying existing pavement layers or between layers of asphalt binder mixes where multiple lifts are required.

(d) Prime Coats can be used on aggregate base prior to paving for better bonding and to act as water proofing of the aggregate base.

(e) Leveling Courses are used to fill and level surface irregularities and ruts before placing overlays.

Topic 603 – Types of Pavement Projects

603.1 New Construction

New construction is the building of a new facility. This includes new roadways, interchanges or grade separation crossings, and new parking lots or safety roadside rest areas.

603.2 Widening

Widening projects involve the construction of additional width to improve traffic flow and increase capacity on an existing highway facility. Widening may involve adding lanes (including bus or bicycle lanes), shoulders, pullouts for maintenance/transit traffic; or widening existing lane, shoulder or pullouts.

It is often not cost-effective or desirable to widen a highway without correcting for bad ride and major structural problems in adjacent pavements when that work is needed. Therefore, on widening projects such as lane/shoulders additions, auxiliary lanes, climbing or passing lanes, etc., the existing adjacent pavement condition should be investigated to determine if rehabilitation or pavement preservation is warranted. If warranted, combining rehabilitation or pavement preservation work with widening is strongly encouraged. Combining widening with work on existing pavement can minimize traffic delay and long-term costs. For example, grinding the adjoining rigid pavement lane next to the proposed widening can improve constructability and provide a smoother pavement surface for the widening. For flexible pavement projects, a minimum of 45 mm overlay over the widening and existing pavement should be used to eliminate pavement joints which are susceptible to water intrusion and early fatigue failure.

Additional guidance and requirements on widening existing facilities, including possible options as well as certain circumstances that may justify adding rehabilitation or pavement preservation work to widening, or deferring it, are discussed in Index 612.3 and the Project Development Procedures Manual (PDPM) Chapter 8, Section 7.

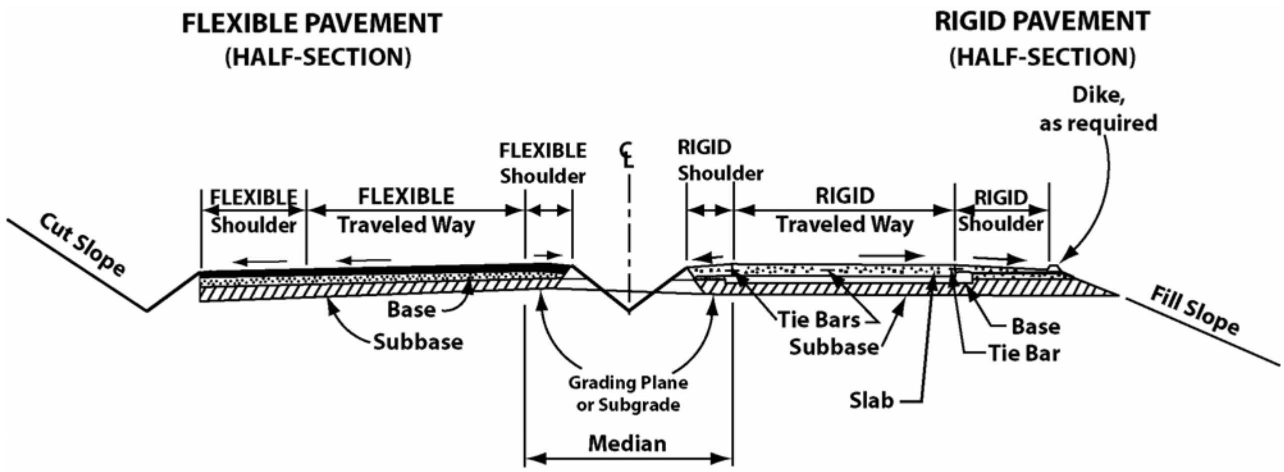
603.3 Pavement Preservation

Pavement Preservation has two main categories or programs:

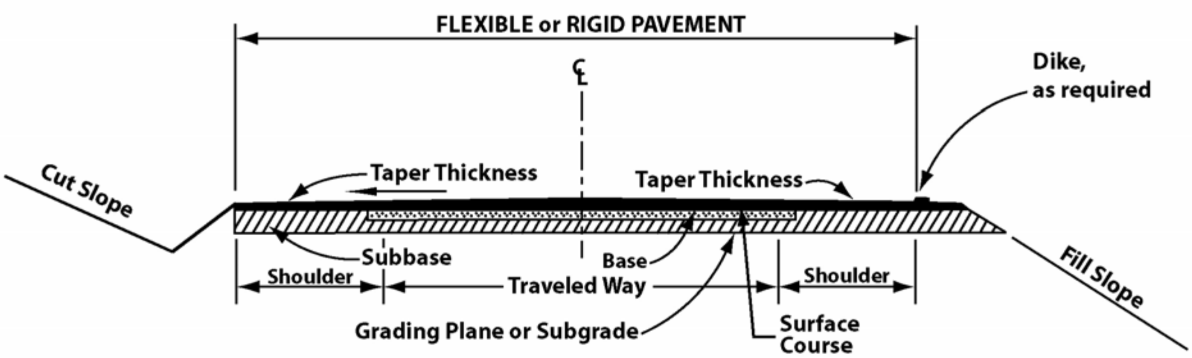
(1) *Preventive Maintenance.* Preventive maintenance projects are used to provide preventive treatments to preserve pavements in good condition. These projects are typically done by Department Maintenance forces or through the Major Maintenance Program. The District Maintenance Engineer typically determines which preventive treatment to apply and when. Examples of preventive maintenance projects include: removal and replacement of a non-structural wearing course (for example, open graded friction courses); thin non-structural overlays less than or equal to 25 mm (or 30 mm when needed to enhance compaction in colder

Figure 602.1

Basic Pavement Layers of the Roadway



DIVIDED HIGHWAYS



UNDIVIDED HIGHWAYS

Notes:

1. These illustrations are only to show nomenclature and are not to be used for geometric cross section details. For these, see Chapter 300.

2. Pavement drainage design, both on divided and undivided highways, are illustrated and discussed under Chapter 650.

3. Only flexible and rigid pavements shown. Composite pavements are the same as rigid pavements with a flexible layer overlay.

4. See Index 626.2 for criteria for when and how to use flexible or rigid shoulders.

termperatures); 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 preventive 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 2.68 m/km) as determined by the Pavement Condition Survey (PCS) and the Pavement Management System (PMS). Ride improvement and preservation of the 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 a 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 of 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 60 mm (75 mm if International Roughness Index >2.68 m/km).
- 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 75 mm.
- Removal and replacement of more than 75 mm 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 reserve 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 standards are also considered a rehabilitation and not reconstruction. Storm or earthquake damage repair (i.e., catastrophic) also are not considered reconstruction projects.

Pavement reconstruction projects are to follow the same standards as new construction found in this manual unless noted otherwise.

603.6 Temporary Pavements and Detours

Temporary pavements and detours are constructed to temporarily carry traffic anticipated during construction. These types of pavements should be engineered using the standards and procedures for new construction except where noted otherwise.

Topic 604 - Roles and Responsibilities

604.1 Roles and Responsibilities for Pavement Engineering

The roles and responsibilities listed below apply only to pavement engineering.

(1) *Pavement Engineer*. The pavement engineer is the engineer who performs pavement calculations, develops pavement structure recommendations, details, or plans. The pavement engineer can be the Project Engineer, District Materials Engineer, District Maintenance Engineer, consultant, or other staff engineer responsible for this task.

(2) *Project Engineer (PE).* The PE is the registered civil engineer in responsible charge of appropriate project development documents (i.e., Project Study Report, Project Report, and PS&E) and coordinates all aspects of project development. The PE is responsible for project technical decisions, engineering quality (quality control), and estimates. This includes collaborating with the District Materials Engineer, District Pavement Advisor and other subject matter experts regarding pavement details and selecting pavement strategy for new and rehabilitation projects. The PE clearly conveys pavement related decisions and information on the project plans and specifications for a Contractor to bid and build the project.

The PE coordinates with the Structures District Liaison Engineer and Division of Engineering Services (DES) staff for the proper selection and engineering of any structure approach system including the adequacy of all drainage ties between the structure approach drainage features and other new or existing drainage facilities. The PE should contact the Structures District Liaison Engineer as early as possible in the project development process to facilitate timely review and project scheduling.

(3) *District Materials Engineer (DME).* The DME is responsible for materials information for pavement projects in the district. The District Materials Unit is responsible for conducting or reviewing the findings of a preliminary soils and other materials investigation to evaluate the quality of the materials available for constructing the project. The DME prepares or reviews the Materials Report for each project; provides recommendations to and in continuous consultation with the Project Engineer throughout planning and design, and with the PE and Resident Engineer during construction; and coordinates Materials information with the Department functional units, Material Engineering and Testing Services (METS), Headquarters functional units, local agencies, industry, and consultants.

(4) *District Pavement Advisor (DPA).* The DPA manages and coordinates overall pavement strategies for the District. They are primarily involved in pavement management such as identifying future pavement preservation, rehabilitation, and reconstruction needs, and prioritizing pavement projects to meet those needs. The DPA establishes pavement projects and reviews planning documents prepared by the PE for consistency with overall District and statewide goals for pavements. The District Pavement Advisor is typically either the District Maintenance Engineer or another individual within District Maintenance.

(5) *Pavement Program Steering Committee (PPSC).* The PPSC provides leadership and commitment to ensure safe, effective, and environmentally sensitive highway pavements that improve mobility across California. The PPSC is responsible for conveying clear direction and priorities on pavement initiatives, policies, and standards that reflect departmental goals; and for the implementation of pavement policies, standards, and specifications. Members of the PPSC include Headquarters Division Chiefs (Construction, Design, Engineering Services, Maintenance, Project Management, and Research & Innovation) and three District Directors.

(6) *Pavement Standards Team (PST).* The PST is a multifunctional group consisting of METS, Design, Construction, Maintenance, Research and Innovation, Office Engineer, Project Manager for the team, selected District representatives and the FHWA. The PST provides policies, procedures, and practices regarding pavement engineering, construction, maintenance, and rehabilitation to ensure consistency and quality of the pavement structure throughout the State. The PST also develops and maintains all pavement related standards, specifications, and procedures; approves nonstandard specifications, and provides recommendations to the PPSC.

(7) *Division of Design (DOD).* The DOD is responsible for statewide standards and guidelines in the project engineering process. The DOD Office of Pavement Design (OPD) is responsible for communicating and maintaining pavement engineering standards, policies, procedures, and practices that are used statewide.

(8) *Materials Engineering and Testing Services (METS).* METS is a subdivision of the Division of Engineering Services (DES), which is responsible for conducting laboratory testing, field testing, specialized field inspections, and expert advice on materials and manufactured products. METS provides technical expertise on material properties and products for the development of statewide standards, guidelines, and procedure manuals. METS also works closely with the District Materials Engineers and Resident Engineers to investigate ongoing field and materials issues.

(9) *DES Office of Structure Design (OSD).* The OSD is responsible for selecting the type of structure approach system to be used when the construction or rehabilitation of a structure approach slab is necessary.

(10) *DES Geotechnical Services (DES-GS).* The DES-GS provides the Districts, Structures, and Headquarters with expertise and guidance in soil related investigations and groundwater issues. DES-GS prepares or reviews Geotechnical Design Reports based upon studies and information supplied by the District.

604.2 Other Resources

The following resources provide additional standards and guidance related to pavement engineering. Much of this information can be found on the Department pavement website, see category (5) below.

(1) *Standard Plans.* These are collections of commonly used engineering details intended to provide consistency for contractors, resident engineers and maintenance engineers in defining the scope of work for projects, assist in the biddability of the project contract plans, and assist maintenance in maintaining the facility. The standard plans were developed based on research and field experience and in consultation with industry. Standard plans for pavement must not be altered or modified without the prior written approval of the Chief, Office of Pavement Design. Standard plans for pavements can be found on the Department pavement website.

(2) *Standard Specifications and Standard Special Provisions.* The Standard Specifications provide material descriptions, properties and work quality requirements, contract administration requirements, and measurement and payment clauses for items used in the project. The Standard Special Provisions are additional specification standards used to

modify the Standard Specifications including descriptions, quality requirements, and measurement and payment for the project work and materials. When no Standard Specification or Standard Special Provision exists for new or proprietary items, the Pavement 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 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/oppd/pavement/index.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, AASHTO and other States. On March 10, 2005, the Department committed to develop ME Design as an alternative and possible replacement of current methods. The Department is currently working on the procedures and criteria for performing this analysis. Until the criteria are established and the methodology verified, ME Design will be considered experimental and cannot, at this time, be used to engineer pavements on the State highway system or other roads maintained by the State.

606.4 Proprietary Items

The use of proprietary materials and methods on State highway projects is discussed in Topic 110.10

612.3 Widening

Additional consideration is needed when determining the design life for pavement widening. Factors to consider include the remaining service life of the adjacent pavement, planned future projects (including maintenance and rehabilitation), and future corridor plans for any additional lane widening and shoulders. **The pavement design life for widening projects shall either match the remaining pavement service life of the adjacent roadway (but not less than the project design period as defined in Index 103.2), or the pavement design life values in Table 612.2 depending on which has the lowest life-cycle costs.** Life-cycle cost analysis is discussed further in Topic 619.

When widening a roadway, the existing pavement should be rehabilitated and brought up to the same life expectancy as the new widened portion of the roadway.

612.4 Pavement Preservation

Since pavement preservation projects involve non-structural overlays, seals, grinds, or repairs; they are not engineered to meet a minimum structural design life like other types of pavement projects. Instead, pavement preservation projects, which include preventive maintenance and capital preventive maintenance strategies, are engineered to extend the service life of existing pavements as follows:

(1) *Preventive Maintenance:* Preventive maintenance strategies are intended to extend the service life of an existing pavement structure while it is in good condition. Typically, for preventive maintenance, the added service life can vary from a minimum of 2 years to over 7 years depending on the strategy being used and the condition of the existing pavement.

(2) *Capital Preventive Maintenance:* The strategies used for CAPM projects have been engineered to extend the service life by a minimum of 5 years of pavement that exhibits minor distress and/or triggered ride (International Roughness Index (IRI) greater than 2.68 m/km). Some strategies such as rigid pavement diamond grinding, slab replacement, punchout repairs, and dowel bar retrofit can last at least 10 years.

612.5 Roadway Rehabilitation

For roadways with existing flexible/composite pavement and a current Annual Average Daily Traffic (AADT) of less than 15,000, the minimum pavement design life shall be 20 years. A 40-year pavement design life may be considered and evaluated for flexible/composite pavement projects with a current AADT less than 15,000 at the District's option.

For roadways with existing rigid pavement regardless of AADT, as well as existing flexible/composite pavement with a current AADT of 15,000 or more, life-cycle cost analysis shall be performed comparing a pavement design life of 20 years with a pavement design life of 40 years. The design representing the lower life-cycle cost shall be selected.

Life cycle cost analysis is discussed further in Topic 619.

612.6 Temporary Pavements and Detours

<u>Temporary pavements and detours should be engineered to accommodate the anticipated traffic loading that the pavement will experience during the construction period.</u> The minimum design life for temporary pavements and detours should be no less than the construction period for the project. This period may range from a few months to several years depending on the type, size and complexity of the project.

612.7 Non-Structural Wearing Courses

As described in Index 602.1(5), a non-structural wearing course is used on some pavements to ensure that the underlying layers will be protected from wear and tear from tire/pavement interaction, the weather, and other environmental factors for the intended design life of the pavement. Because non-structural wearing courses are not considered to contribute to pavement structural capacity, they are not expected to meet the same design life criteria as the structural layers. However, when selecting materials, mix designs and thickness of these courses, appropriate evaluation and sound engineering judgment should be used to optimize performance and minimize the need for maintenance of the wearing course and the underlying structural layers. Based on experience, a properly engineered non-structural wearing course placed on new pavement should perform adequately for 10 or more years, and 5 or more years when placed on existing pavement as a part of rehabilitation or preventive maintenance.

Topic 613 - Traffic Considerations

613.1 Overview

Pavements are engineered to carry the truck traffic loads expected during the pavement design life. Truck traffic, which includes buses, trucks and truck-trailers, is the primary factor affecting pavement design life and its serviceability. Passenger cars and pickups are considered to have negligible effect when determining traffic loads.

Truck traffic information that is required for pavement engineering includes projected volume for each of four categories of truck and bus vehicle types by axle classification (2-, 3-, 4-, and 5-axles or more), axle configurations (single, tandem, tridem, and quad), axle loads, and number of load repetitions. This information is used to estimate anticipated traffic loading and performance of the pavement structure. The Department currently estimates traffic loading by using established constants for a 10-, 20-, 30-, or 40-year pavement design life to convert truck traffic data into 80 kN equivalent single axle loads (ESALs). The total projected ESALs during the pavement design life are in turn converted into a Traffic Index (TI) that is used to determine minimum pavement thickness. Another method for estimating pavement loading known as Axle Load Spectra is currently under development by the Department for future use with the Mechanistic-Empirical design procedure.

613.2 Traffic Volume Projections

(1) *Traffic Volume or Loading Data.* In order to determine expected traffic loads on a pavement it is first necessary to determine projected traffic volumes during the design life for the facility.

Traffic volume and loading on State highways can come from vehicle counts and classification, weigh-in-motion (WIM) stations, or the Truck Traffic (Annual Average Daily Truck Traffic) on California State Highways published annually by Headquarters Division of Traffic Operations. Current and projected traffic volume by vehicle classification must be obtained for each project in accordance with the procedures found in this Topic.

Districts typically have established a unit within Traffic Operations or Planning specifically responsible for providing travel forecast information. These units are responsible for developing traffic projections (including truck volumes, equivalent single axle loads, and TIs) used for planning and engineering of State highways in the District. The Project Engineer should coordinate with the forecasting unit in their District early in the project development process to obtain the required traffic projections.

(2) *Design Year Annual Average Daily Truck Traffic (AADTT)*: An expansion factor obtained from the traffic forecasting unit is used to project current AADTT to the design year AADTT for each axle classification (see Table 613.3A). In its simplest form, the expansion factor is a straight-line projection of the current one-way AADTT data. When using the straight-line projection, the truck traffic data is projected to find the AADTT at the midway of the design life. This represents the average one-way AADTT for each axle classification during the pavement design life.

When other than a straight-line projection of current truck traffic data is used for engineering purposes, the procedure to be followed in developing design year traffic projections will depend on travel forecast information for the region. In such cases, the projections require a coordinated effort from the District's Division of Transportation Planning and Traffic Operations, working closely with the Regional Agencies to establish realistic values for truck traffic growth rates based on travel patterns, land use changes, and other socioeconomic factors.

Due to various changes in travel patterns, land use changes, and other socioeconomic factors that may significantly affect design year traffic projections, the TI for facilities with longer service life, such as a 30- or 40-year design life require more effort to determine than for a 10- or 20-year design life. For this reason, the Project Engineer should involve District Transportation Planning and/or Traffic Operations in determining a realistic and appropriate TI for each project early in the project development process. In the absence of 30- or 40-year traffic projection data, 20-year projection data may be extrapolated to 30- and 40-year values by applying the expansion factors.

613.3 Traffic Index Calculation

The Traffic Index (TI) is determined using the following procedures:

(1) *Determine the Projected Equivalent Single Axle Loads (ESALs).* The information obtained from traffic projections and Truck Weight Studies is used to develop 80 kN Equivalent Single Axle Load (ESAL) constants that represent the estimated total accumulated traffic loading for each heavy vehicle (trucks and buses) and each of the four truck types during the pavement design life. Typically, buses are assumed to be included in the truck counts due to their relatively low number in comparison to trucks. However, for facilities with high percentage of buses such as high-occupancy vehicle (HOV) lanes and exclusive bus lanes, projected bus volumes need to be included in the projection used to determine ESALs. The ESAL constants are used as multipliers of the projected AADTT for each truck type to determine the total cumulative ESALs and in turn the Traffic Index (TI) during the design life for the pavement (see Index 613.3(3)). The ESALs and the resulting TI are the same magnitude for both flexible, rigid, and composite pavement alternatives. The current 10-, 20-, 30-, and 40-year ESAL constants are shown in Table 613.3A.

(2) *Lane Distribution Factors.* Truck/bus traffic on multilane highways normally varies by lane with the lightest volumes generally in the median lanes and heaviest volumes in the outside lanes. Buses are also typically found in HOV lanes. For this reason, the distribution of truck/bus traffic by lanes must be considered in the engineering for all multilane facilities to ensure that traffic loads are appropriately distributed. Because of the uncertainties and the variability of lane distribution of trucks on multilane freeways and expressways, statewide lane distribution factors have been established for pavement engineering of highway facilities in California. These lane distribution factors are shown in Table 613.3B.

(3) *Traffic Index (TI).* The Traffic Index (TI) is a measure of the number of ESALs expected in the traffic lane over the pavement design life of the facility. The TI does not vary linearly with the ESALs but rather according to the following exponential formula and the values presented in Table 613.3C. The TI is determined to the nearest 0.5.

$$TI = 9.0 \times \left(\frac{(ESAL \times LDF)}{10^6} \right)^{0.119}$$

Where:

TI = Traffic Index

ESAL = Total number of cumulative 80 kN Equivalent Single Axle Loads

LDF = Lane Distribution Factor (see Table 613.3B)

Index 613.4 contains additional requirements and considerations for determining projected traffic loads.

613.4 Axle Load Spectra

(1) *Development of Axle Load Spectra.* Axle load spectra is an alternative method of measuring heavy vehicle loads that is currently under development for the future mechanistic-empirical design method. Axle load spectra is

Table 613.3A
ESAL Constants

Vehicle Type (By Axle Classification)	10-Year Constants	20-Year Constants	30-Year Constants	40-Year Constants
2-axle trucks or buses	690	1380	2070	2760
3-axle trucks or buses	1840	3680	5520	7360
4-axle trucks	2940	5880	8820	11760
5 or more-axle trucks	6890	13780	20670	27560

Table 613.3B
Lane Distribution Factors for Multilane Highways

Number of Mixed Flow Lanes in One Direction	Factors to be Applied to Projected Annual Average Daily Truck Traffic (AADTT)			
	Mixed Flow Lanes (see Notes 1, 2, 3 & 4)			
	Lane 1	Lane 2	Lane 3	Lane 4
One	1.0	-	-	-
Two	1.0	1.0	-	-
Three	0.2	0.8	0.8	-
Four	0.2	0.2	0.8	0.8

NOTES:
1. Lane 1 is next to the centerline or median.
2. For more than four lanes in one direction, use a factor of 0.8 for the outer two lanes plus any auxiliary/collector lanes, use a factor of 0.2 for other mixed flow through lanes.
3. For HOV lanes, use a factor of 0.2; however, the TI should be no less than 10 for a 20-year, or 11 for a 40-year pavement design life.
4. For lanes devoted exclusively to buses and/or trucks, use a factor of 1.0 based on projected AADTT of mixed-flow lanes for auxiliary and truck lanes, and a separate AADTT based on expected bus traffic for exclusive bus lanes.

Table 613.3C
Conversion of ESAL to Traffic Index

ESAL [1]	TI [2]	ESAL [1]	TI [2]
4710		6 600 000	
	5.0		11.5
10 900		9 490 000	
	5.5		12.0
23 500		13 500 000	
	6.0		12.5
47 300		18 900 000	
	6.5		13.0
89 800		26 100 000	
	7.0		13.5
164 000		35 600 000	
	7.5		14.0
288 000		48 100 000	
	8.0		14.5
487 000		64 300 000	
	8.5		15.0
798 000		84 700 000	
	9.0		15.5
1 270 000		112 000 000	
	9.5		16.0
1 980 000		144 000 000	
	10.0		16.5
3 020 000		186 000 000	
	10.5		17.0
4 500 000		238 000 000	
	11.0		17.5 [3]
6 600 000		303 000 000	

Notes:

(1) For ESALs less than 5000 or greater than 300 million, use the TI equation to calculate design TI, see Index 613.3(3).

(2) The determination of the TI closer than 0.5 is not justified. No interpolations should be made.

(3) For TI's greater than 17.5, use the TI equation, see Index 613.3(3).

a representation of normalized axle load distribution developed from weigh-in-motion (WIM) data for each axle type (single, tandem, tridem, and quad) and truck class (FHWA vehicle classes 4 through 13). Axle load spectra do not involve conversion of projected traffic loads into equivalent single axle loads (ESALs), instead traffic load applications for each truck class and axle type are directly characterized by the number of axles within each axle load range.

In order to accurately predict traffic load related damage on a pavement structure, it is important to develop both spatial and temporal axle load spectra for different truck loadings and pavements. The following data is needed to develop axle load spectra:

- Truck class (FHWA class 4 for buses through class 13 for 7+ axle multi-trailer combinations)
- Axle type (single, tandem, tridem, and quad)
- Axle load range for each axle type and truck class (13 to 453 kN)
- The number of axle load applications within each axle load range by axle type and truck class
- The percentage of the total number of axle applications within each axle load range with respect to each axle type, truck class, and year of data. These are the normalized values of axle load applications for each axle type and truck class

The aforementioned data are obtained from traffic volume counts and WIM data for vehicle classification, and axle type and weight. Traffic counts and WIM stations should be deployed widely to ensure that projected volume estimates for each vehicle class and axle type are in line with the actual volumes and growth rates.

(2) *Use of Axle Load Spectra in Pavement Engineering:* Pavement engineering calculations using axle load spectra are generally more complex than those using ESALs or traffic index (TI) because loading cannot be reduced to one equivalent number. However, the load spectra approach of quantifying traffic loads offers a more practical and realistic representation of traffic loading than using TI or ESALs. Due to its better performance modeling, axle load spectra will be used in the Mechanistic-Empirical (M-E) design method currently under development to evaluate traffic loading over the design life for new and rehabilitated pavements. This information will be used to validate original pavement design loading assumptions, and to continuously monitor pavement performance given the loading spectrum. Axle load spectral data will also be used to facilitate effective and pro-active deployment of maintenance efforts and in the development of appropriate strategies to mitigate sudden and unexpected pavement deterioration due to increased volumes or loading patterns.

In this edition of the Highway Design Manual, axle load spectra are not used to engineer pavements.

613.5 Specific Traffic Loading Considerations

(1) *Traveled Way*

(a) Mainline Lanes. Because each lane for a multilane highway with 3 or more lanes in each direction may have a different load distribution factor (see Table 613.3B), multiple TIs may be generated for the mainline lanes which can result in different pavement thickness for each lane. Such a design with different thickness for each lane would create complications for constructing the pavement. Therefore, the decision to use a single or multiple TI's for the pavement engineering of mainline lanes for a multilane highway with 3 or more lanes in each direction should be based on a thorough consideration of constructibility issues discussed in Index 618.2 together with sound engineering judgment. If one TI is used, it should be the one that produces the most conservative pavement structure.

(b) Freeway Lanes. TI for new freeway lanes, including widening, auxiliary lanes, and high-occupancy vehicle (HOV) lanes, should be the greater of either the calculated value, 10.0 for a 20-year pavement design life, or 11.0 for a 40-year pavement design life. For roadway rehabilitation projects, use the calculated TI.

(c) Ramps and Connectors:

1. Connectors. AADTT and TI's for freeway-to-freeway connectors should be determined the same way as for mainline traffic.

2. Ramps to Weigh Stations. Pavement structure for ramps to weigh stations should be engineered using the mainline ESALs and the load distribution factor of 1.0 for exclusive truck lanes as noted in Table 613.3B.

3. Other Ramps. Estimating future truck traffic on ramps is more difficult than on through traffic lanes. It is typically more difficult to accurately forecast ramp AADTT because of a much greater impact of commercial and industrial development on ramp truck traffic than it is on mainline truck traffic.

   If reliable truck traffic forecasts are not available, ramps should be engineered using the 10-, 20-, and 40-year TI values given in Table 613.5A for light, medium, and heavy truck traffic ramp classifications. Design life TI should be the greater of the calculated TI or the TI values in Table 613.5A.

The three ramp classifications are defined as follows:

- Light Traffic Ramps - Ramps serving undeveloped or residential areas with light to no truck traffic predicted during the pavement design life.
- Medium Traffic Ramps - Ramps in metropolitan areas, business districts, or where increased truck traffic is likely to develop because of anticipated commercial development within the pavement design life
- Heavy Traffic Ramps - Ramps that will or currently serve industrial areas, truck terminals, truck stops, and/or maritime shipping facilities.

The final decision on ramp truck traffic classification rests with the District.

Table 613.5A
Traffic Index (TI) Values for Ramps and Connectors

Ramp Truck Traffic Classification	Minimum Traffic Index (TI)		
	10–Yr Design Life (1)	20–Yr Design Life	40–Yr Design Life (1)
Light	8.0	8.0	9.0
Medium	9.0	10.0	11.0
Heavy	11.0	12.0	14.0

Note:

(1) Based on straight line extrapolation of 20-year ESALs.

(2) Shoulders

(a) New Construction and Reconstruction. Because shoulders do not typically carry repeated traffic loads like traffic lanes, the pavement structure for the shoulder is engineered based on the traffic loads of the adjacent traffic lane. Preferably, all new or reconstructed shoulders should match the pavement structure of the adjacent traffic lane, except when the thickness of the flexible surface course can vary to account for the difference in cross slope between the traveled way and the shoulder. This strategy has been the most effective over time in optimizing the performance of the shoulders and minimizing maintenance and repair. Besides improved performance, new or reconstructed shoulders that match the pavement structure of the adjacent traffic lane have the following additional benefits:

- Simplify the contractor's operation which leads to reduced working days, fewer material needs, and lower unit prices.
- Provide versatility in using the shoulders as temporary detours for construction or maintenance activities in the future.
- Make it easier and more cost-effective to convert into a traffic lane as part of a future widening.

In some cases, it may not be practical to match the pavement structure of the adjacent traffic lane. Such situations are determined or agreed to by the District on a case-by-case basis provided the minimum requirements stated in this manual are met.

At a minimum, new or reconstructed shoulders shall be engineered using the same TI as the adjacent traffic lane when any of the following conditions apply:

- **The shoulder width is 1.5 meters or less.**
- **Where there are sustained (greater than 1.6 km in length) grades of over 4 percent without a truck climbing lane.**
- **The shoulders are adjacent to exclusive truck or bus only lanes, or weigh station ramps.**

For all other cases, the minimum TI for the shoulder shall match the TI of the adjacent traffic lane for the first 0.6 m of the shoulder width measured from the edge of traveled way. For the remaining width of the shoulder, the TI shall be no less than 2 percent of the projected ESALs of the adjacent traffic lane or a TI of 5, whichever is greater.

Note that although using a thinner shoulder pavement structure than the traveled way requires less material and may appear to reduce construction costs, the added costs of time and labor to the Contractor to build the "steps" between the traveled way and shoulder can offset the savings from reduced materials.

(b) Future Conversion to Lane. On new facilities, if the future conversion of the shoulder to a traffic lane is within the pavement design life, the shoulder pavement structure should be equal to that of the adjacent traveled way.

If a decision has been made to convert an existing shoulder to a portion of a traffic lane, a deflection study must be performed to determine the structural adequacy of the in place shoulder pavement structure. The condition of the existing shoulder must also be evaluated for undulating grade, rolled-up hot mix asphalt and the rigid pavement joint, surface cracking, raveling, brittleness, oxidation, etc.

The converted facility must provide a roadway that is structurally adequate for the proposed pavement design life. This is necessary to eliminate or minimize the likelihood of excessive maintenance or rehabilitation being required in a relatively short time because of inadequate structural strength and deterioration of the existing hot mix asphalt.

(c) Tracking Width Lines. For projects where the tracking width lines are shown to encroach onto paved shoulders, the shoulder pavement structure must be engineered to sustain the weight of the design vehicle. If curb and gutter are present and any portion of the gutter pan is likewise encroached, the gutter pan must be engineered to match the adjacent shoulder pavement structure. See Topic 404 for design vehicle guidance.

(d) Medians. When a median is 4.2 meters wide or less, the median pavement structure should be equivalent to the adjacent lanes. See Index 305.5 for further paved median guidance.

(e) Maintenance and Rehabilitation. Traffic Index is not a consideration in a shoulder maintenance or rehabilitation strategy unless the shoulder will be used to temporarily detour traffic or is expected to carry traffic after construction. In such situations, the existing shoulder pavement structure should be checked for structural adequacy. If the shoulder is not structurally adequate, it should be removed and replaced using the procedures and standards described in Index 613.5(2)(a) for new construction and reconstruction. Regardless of whether or not TI is considered, shoulder maintenance or rehabilitation repairs in the existing shoulder are often necessary and should be done to assure that the shoulder pavement will meet the performance requirements.

(3) *Intersections.* Future AADTT and TI's for intersections should be determined the same way as for mainline traffic, but with special attention to truck and bus traffic behavior to determine the loading patterns and select the most appropriate materials. The limits for engineering pavement at an intersection should include intersection approaches and departures, to the greater of the following distances:

- For signalized intersections, the limits of the approach should extend past the furthest set of signal loop detectors where trucks do the majority of their braking; or
- For stop controlled intersections the limits for the approach should be long enough to cover the distance trucks will be braking and stopping either at the stop bar or behind other trucks and vehicles; or
- 30 meters.

The limits for the intersection departures should match the limits of the approach in the opposing lane to address rutting caused by truck acceleration.

For further assistance on this subject, contact either your District Materials Engineer, or Headquarters Division of Design – Office of Pavement Design.

(4) *Roadside Facilities.* The pavement for safety roadside rest areas, including parking lots, should meet or exceed the TI requirements found in Table 613.5B for a 20-year pavement design life for new/reconstructed or rehabilitated pavements.

Table 613.5B
Minimum TI's for Safety Roadside Rest Areas

Facility Usage	Minimum TI (20-Year)
Truck Ramps & Roads	8.0 (1)
Truck Parking Areas	6.0 (1)
Auto Roads	5.5
Auto Parking Areas	5.0

Note:

(1) For safety roadside rest areas next to all Interstates and those State Routes with AADTT greater than 15000 use Table 613.5A medium truck traffic for truck ramps, truck roads, and a minimum TI of 9.0 for truck parking areas.

Topic 614 - Soil Characteristics

614.1 Engineering Considerations

California is a geologically active state with a wide variety of soil types throughout. Thorough understanding of the native soils in a project area is essential to properly engineer or update a highway facility.

Subgrade is the natural soil or rock material underlying the pavement structure. Unlike concrete and steel whose characteristics are fairly uniform, the engineering properties of subgrade soils may vary widely over the length of a project.

Pavements are engineered to distribute stresses imposed by traffic to the subgrade. For this reason, subgrade condition is a principal factor in selecting the pavement structure. Before a pavement is engineered, the structural quality of the subgrade soils must be evaluated to ensure that it has adequate strength to carry the predicted traffic loads during the design life of the pavement. The pavement must also be engineered to limit the expansion and loss of density of the subgrade soil.

614.2 Unified Soil Classification System (USCS)

The USCS classifies soils according to their grain size distribution and plasticity. Therefore, only a sieve analysis and Atterberg limits (liquid limit, plastic limit and plasticity index) are necessary to classify a soil in this system. Based on grain size distribution, soils are classified as either (1) coarse grained (more than 50% retained on the 0.075 mm – No. 200 sieve), or (2) fine grained (50% or more passes the 0.075 mm – No. 200 sieve). Coarse grained soils are further classified as gravels (50% or more of coarse fraction retained on the 4.75 mm – No. 4 sieve) or sands (50% or more of coarse fraction passes the 4.75 mm – No. 4 sieve); while fine grained soils are classified as inorganic or organic silts and clays and by their liquid limit (equal to or less than 50%, or greater than 50%). The USCS also includes peat and other highly organic soils, which are compressible and not recommended for roadway construction. Peat and other highly organic soils should be removed wherever possible prior to placing the pavement structure.

The USCS based on ASTM D 2487 is summarized in Table 614.2.

614.3 California R-Value

The California R-value is the measure of resistance to deformation of the soils under wheel loading and saturated soil conditions. It is used to determine the bearing value of the subgrade. Determination of R-value for subgrade is provided under California Test Method (CTM) 301. Typical R-values used by the Department range from five for very soft material to 80 for treated base material.

The California R-value is determined based on the following separate measurements under CTM 301:

- The exudation pressure test determines the thickness of cover or pavement structure required to prevent plastic deformation of the soil under imposed wheel loads.
- The expansion pressure test determines the pavement thickness or weight of cover required to withstand the expansion pressure of the soil.

Because some soils, such as coarse grained gravel and sands, may exhibit a higher California R-value test result than would normally be required for pavement design, the California R-value for subgrade soils used for pavement design should be limited to no more than 50 unless agreed to otherwise by the District Materials Engineer. Local experience with these soils should govern in assigning R-value on subgrade.

The California R-value of subgrade within a project may vary substantially but cost and constructability should be considered in specifying one or several California R-value(s) for the project. Engineering judgment should be exercised in selecting appropriate California R-values for the project to assure a reasonably "balanced design" which will avoid excessive costs resulting from over conservatism. The following should be considered when selecting California R-values for a project:

- If the measured California R-values are in a narrow range with some scattered higher values, the lowest California R-value should be selected for the pavement design.
- If there are a few exceptionally low California R-values and they represent a relatively small volume of subgrade or they are concentrated in a small area, it may be more cost effective to remove or treat these materials.
- Where changing geological formations and soil types are encountered along the length of a project, it may be cost-effective to design more than one pavement structure to accommodate major differences in R-values that extend over a considerable length. Care should be exercised to avoid many

Table 614.2
Unified Soil Classification System (from ASTM D 2487)

Major Classification Group	Sub-Groups		Classification Symbol	Description
Coarse Grained Soils More than 50% retained on the 0.075 mm (No. 200) sieve	**Gravels** 50% or more of coarse fraction retained on the 4.75 mm (No. 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of coarse fraction passes the 4.75 mm (No. 4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine Grained Soils More than 50% passes the 0.075 mm (No. 200) sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

Prefix: G = Gravel, S = Sand, M = Silt, C = Clay, O = Organic
Suffix: W = Well Graded, P = Poorly Graded, M = Silty, L = Clay, LL < 50%, H = Clay, LL > 50%

variations in the pavement structure that may result in increased construction costs that exceed potential materials cost savings.

614.4 Expansive soils

With an expansive subgrade (Plasticity Index greater than 12), special engineering or construction considerations will be required. Engineering alternatives, which have been used to compensate for expansive soils, are:

(a) Treating expansive soil with lime or other additives to reduce expansion in the presence of moisture. Lime is often used with highly plastic, fine-grained soils. When mixed and compacted, the plasticity and swelling potential of clay soils are reduced and workability increased, as lime combines with the clay particles. It also increases the California R-value of the subgrade. Soil treated with lime is considered to be lime treated subbase. Lime treated subbase is discussed further in Chapter 660.

(b) Replacing the expansive material with a non-expansive material to a depth where the seasonal moisture content will remain nearly constant.

(c) Providing a pavement structure of sufficient thickness to counteract the expansion pressure.

(d) Utilizing two-stage construction by placing a base or subbase to permit the underlying material to expand and stabilize before placing leveling and surface courses.

(e) Stabilizing the moisture content by minimizing the access of water through surface and subsurface drainage and the use of a waterproof membrane (i.e., geomembrane, asphalt saturated fabric, or rubberized asphalt membrane).

(f) Relocating the project alignment to a more suitable soil condition.

Treatment (e) is considered to be the most effective approach if relocation is not feasible such as in the San Joaquin Delta. The District Materials Engineer determines which treatment(s) is/are practical.

The California R-value of the subgrade can be raised above 10 by treatment to a minimum depth of 200 mm with an approved stabilizing agent such as lime, cement, asphalt, or fly ash. Native soil samples should be taken, treated, and tested to determine the California R-value for the treated subgrade. For pavement structure design, the maximum California R-value that can be specified for treated subgrade regardless of test results is 40. Treating the subgrade does not eliminate or reduce the required aggregate subbase for rigid or composite pavements in the rigid pavement catalog (see Topic 623). With HMA, treated subgrade can be substituted for all or part of the required aggregate subbase layer. Since aggregate subbase has a gravel factor (G_f) of 1.0, the actual thickness and the gravel equivalent (GE) are equal. When the treated subgrade is substituted for aggregate subbase for flexible pavements, the actual thickness of the treated subgrade layer is obtained by dividing the GE by the appropriate G_f. The G_f is determined based on unconfined compressive strength (UCS) of the treated material as follows:

$$G_f = 0.9 + \frac{UCS}{6.9}$$

This equation is only valid for UCS of 2.07 MPa or more. The minimum gravel factor G_f should be 1.2. The maximum G_f allowed using this equation is 1.7. Because the treatment of subgrade soil may be less expensive than the base material, the calculated base thickness can be reduced and the treated subgrade thickness increased because of cost considerations. The base thickness is reduced by the corresponding gravel equivalency provided by the lime treated subgrade soil or subbase. The maximum thickness of lime treated subgrade is limited to 600 mm.

Rigid or composite pavement should not be specified in areas with expansive soils unless the pavement has been adequately treated to address soil expansion. Flexible pavement may be specified in areas where expansive soils are present with the understanding that periodic maintenance would be required.

The District Materials Engineer should be contacted to assist with the selection of the most appropriate method to treat expansive soils for individual projects. Final decision as to which treatment to use rests with the District.

614.5 Subgrade Enhancement Geotextile (SEG)

The placement of subgrade enhancement geotextile (SEG), formerly called subgrade enhancement fabric (SEF), below the pavement will provide subgrade enhancement by bridging soft areas and providing a separation between soft subgrade fines susceptible to pumping and high quality subbase or base materials. One weak subgrades, the use of SEG can provide for stabilization (the coincident function of separation and reinforcement). As the soft soil undergoes deformation, properly placed geotextile when stretched will develop tensile stress. Locations that may require placement of SEG include areas with the following soil characteristics:

- Poor (low strength) soils which are classified in the unified soil classification system (USCS) as sandy clay (SC), silty clay (CL), high plastic clay (CH), silt (ML), high plasticity or micaceous silt (MH), organic silt (OL), organic clay (OH), and peat & mulch (PT).
- Low undrained shear strength (equivalent to California R-value <20).
- High water table, and high soil sensitivity.

Subgrade soils with R-value <20 are considered poor or weak soils and require SEG to provide reinforcement as the primary function and separation as the secondary function. However, pavements constructed over subgrade soils with R-value up to 40 can especially benefit from separation if the soil contains an appreciable amount of fines, depending on type and treatment of the base layer. The SEG when placed with aggregate subbase provides a working platform for access of construction equipment, mainly on subgrades with R-values of 5 to 10.

The use of SEG on weak subgrades (with R-value <20) can raise the effective R-value of such soils to 20. Therefore, the benefit of using SEG on such weak soils can be realized though using thinner aggregate bases or subbases in flexible pavement design. Likewise, SEG can also affect the design of rigid pavements by providing a stronger subgrade system.

The method of determining the functions realized from the use of SEG and the selection of the appropriate properties of the SEG based on project specifics are explained in the "Subgrade Enhancement Getotextile Guide" on the Department Pavement website.

614.6 Other Considerations

(1) *Fill.* Because the quality of excavated material may vary substantially along the project length, the pavement design over a fill section should be based on the minimum California R-value or unified soil classification of the material that is to be excavated as part of the project. If there is any excavated material that should not be used, it should be identified in the Materials Report and noted as appropriate in the PS&E.

(2) *Imported Borrow.* Imported borrow is used in the construction of embankments when sufficient quantity of quality material is not available. The pavement design should be based on the minimum California R-value of imported borrow or excavated fill material on the project. When imported borrow of desired quality is not economically available or when all of the earthwork consists of borrow, the California R-value specified for the borrow becomes the design R-value. Since no minimum California R-value is required by the Standard Specifications for imported borrow, a minimum R-value for the imported borrow material placed within 1.2 m of the grading plane must be specified in the Materials Report and in the project plans.

(3) *Compaction.* Compaction is densification of the soil by mechanical means. The Standard Specifications require no less than 95 percent relative compaction be obtained for a minimum depth of 800 mm below finished grade for the width of the traveled way and auxiliary lanes plus 1 m on each side. The 800 mm depth of compaction should not be waived for the traveled way, auxiliary lanes, and ramps on State highways.

These specifications sometimes can be waived by special provision with approval from the District Materials Engineer, when any of the following conditions apply:

- A portion of a local road is being replaced with a stronger pavement structure.
- Partial-depth reconstruction is specified.
- Existing buried utilities would have to be moved.
- Interim widening projects are required on low-volume roads, intersection channelization, or frontage roads.

Locations where the 800 mm compaction depth is waived must be shown on the typical cross sections of the project plan. If soft material below this depth is encountered, it must be removed and replaced with suitable excavated material, imported borrow or subgrade enhancement fabric. Location(s) where the Special Provisions apply should be shown on the typical cross section(s).

Topic 615 - Climate

The effects that climate will have on pavement must be considered as part of pavement engineering. Temperatures will cause pavements to expand and contract creating pressures that can cause pavements to buckle or crack. Binders in flexible pavements will also become softer at higher temperatures and more brittle at colder temperatures. Precipitation can increase the potential for water to infiltrate the base and subbase layers, thereby resulting in increased susceptibility to erosion and weakening of the pavement structural strength. In freeze/thaw environments, the expansion and contraction of water as it goes through freeze and thaw cycles, plus the use of salts, sands, chains, and snow plows, create additional stresses on pavements. Solar radiation can also cause some pavements to oxidize. To help account for the effects of various climatic conditions on pavement performance, the State has been divided into the following nine climate regions.

- North Coast
- Central Coast
- South Coast
- Low Mountain
- High Mountain
- South Mountain
- Inland Valley
- Desert
- High Desert

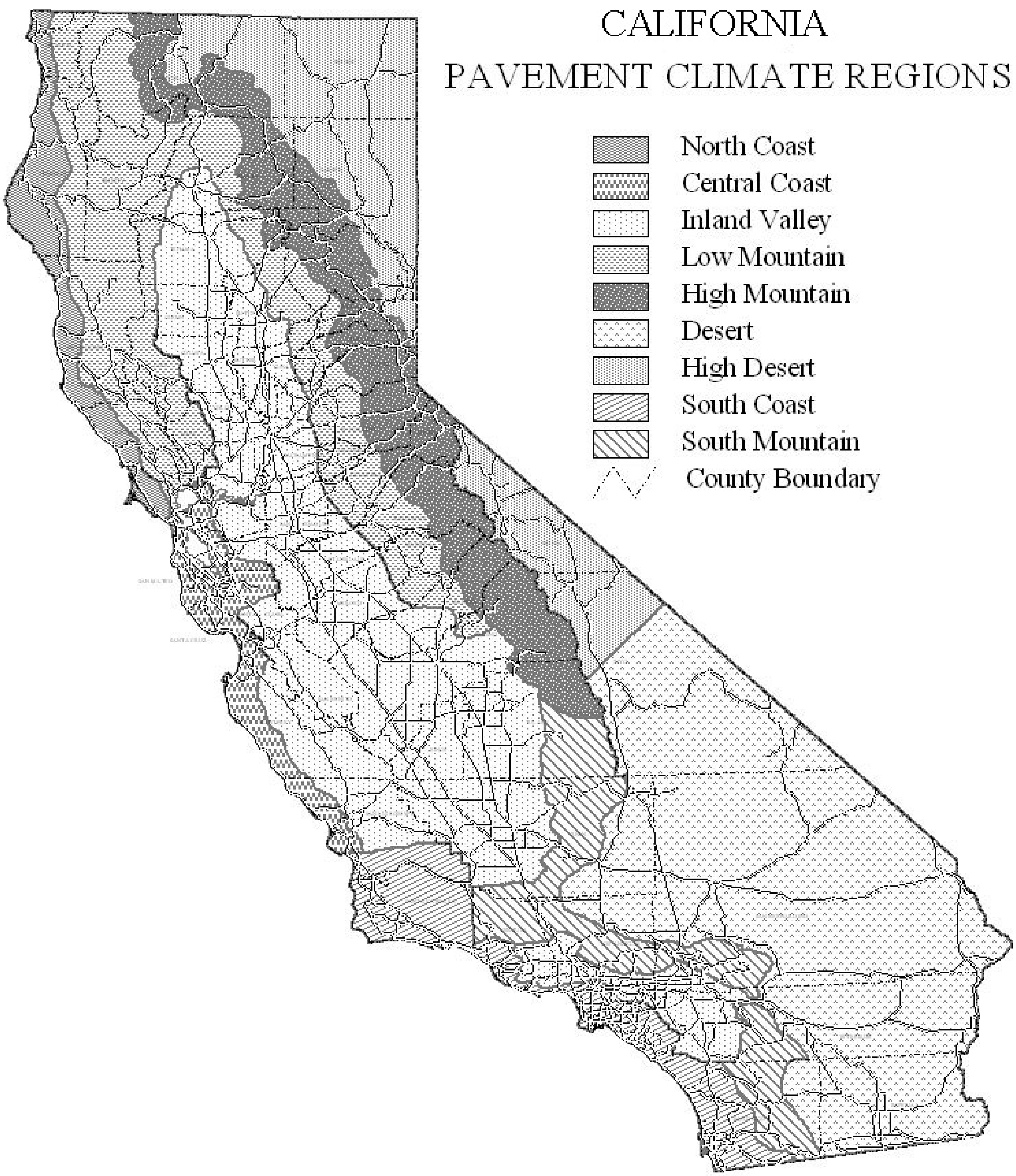
Figure 615.1 provides a representation of where these regions are. A more detailed map along with a detailed list of where State routes fall within each climate region can be found on the Department pavement website.

In conjunction with this map, designs, standards, plans, and specifications have been and are being developed to tailor pavement standards and practices to meet each of these climatic conditions. The standards and practices found in this manual, the Standard Plans, Standard Specification, and Special Provisions should be considered as the minimum requirements to meet the needs of each climate region. Districts may also have additional requirements based on their local conditions. Final decision for the need for any requirements that exceed the requirements found in this manual, the Standard Plans, Standard Specifications, and Standard Special Provisions rests with the District.

Topic 616 - Existing Pavement Type and Condition

The type and condition of pavement on existing adjacent lanes or facilities should be considered when selecting new pavement structures or rehabilitation/preservation strategies. The selection process and choice made by the engineer is influenced by their experience and knowledge of existing facilities in the immediate area that have given adequate service. Providing continuity of existing pavement type will also ensure consistency in maintenance operations.

Figure 615.1
Pavement Climate Regions



NOTE: Map is shown for reference only.
See the Department Pavement website for the detailed map to use.

In reviewing existing pavement type and condition, the following factors should be considered:

- Type of pavement on existing adjacent lanes or facilities
- Performance of similar pavements in the project area
- Corridor continuity
- Maintaining or changing grade profile
- Existing pavement widening with a similar material
- Existing appurtenant features (median barriers, drainage facilities, curbs and dikes, lateral and overhead clearances, and structures which may limit the new or rehabilitated pavement structure.)

Topic 617 - Materials

617.1 Availability of Materials

The availability of suitable materials such as subbase and base materials, aggregates, binders, and cements for pavements should be considered in the selection of pavement type. The availability of commercially produced mixes and the equipment capabilities of area contractors may also influence the selection of pavement type, particularly on small widening, reconstruction or rehabilitation projects. Materials which are locally available or require less energy to produce and transport to the project site should be used whenever possible.

617.2 Recycling

The Department encourages and seeks opportunities to utilize recycled materials in construction projects whenever such materials meet the minimum engineering standards and are economically viable. Accordingly, consideration should be given on every project to use materials recycled from existing pavements as well as other recycled materials such as scrap tires. Existing pavements can be recycled for use as subbase and base materials, or as a partial substitute for aggregate in flexible surface course for rehabilitation or reconstruction projects. The decision to use recycled materials however should be made on a case-by-case basis based on a thorough evaluation of material properties, performance experience in prior projects, benefit/cost analysis, and engineering judgment. Additional information on use of recycled pavements is available in Index 110.11 and on the Department pavement website.

Candidates for recycling flexible pavement surface courses are those with uniform asphalt content. The existence of heavy crack-sealant, numerous patches, open-graded friction course, and heavy seal coats make the new recycled hot mix asphalt design inconsistent thereby resulting in mix properties that are more difficult to control. To avoid this problem when it occurs and still use the recycle option, for flexible pavement, a minimum of 25 mm should be milled off prior to the recycling operation. Light crack sealing (less than 5 percent of the pavement) or a uniform single seal coat will not influence the pavement engineering sufficiently to require milling.

The Department has established a minimum mill depth of 45 mm for recycling flexible pavement surface courses. Since existing surface course thickness will have slight variations, the recycling strategy should leave at least the bottom 45 mm of the existing flexible surface course in place. This is to insure the milling machine does not loosen base material and possibly contaminate the recycled material. As mentioned in Index 110.11(2), recycling of existing hot mix asphalt must be considered, in all cases, as an alternative to placing 100 percent new hot mix asphalt.

Topic 618 - Maintainability and Constructibility

618.1 Maintainability

Maintainability is the ability of a highway facility to be restored in a timely and cost-effective way with minimal traffic exposure to the workers and minimal traffic delays to the traveling public. It is an important factor in the selection of pavement type and pertinent appurtenances. Maintainability issues should be considered throughout the project development process to ensure that maintenance needs are adequately addressed in the engineering and construction of the pavement structure. For example, while a project may be constructible and built in a timely and cost-effective manner, it may create conditions requiring increased worker exposure and increased maintenance effort that is more expensive and labor intensive to maintain. Another example is the pavement drainage systems that need frequent replacement and often do not provide access for cleanout.

Besides the minimum considerations for the safety of the public and construction workers found in this manual, the Standard Specifications, and other Department manuals and guidance, greater emphasis should also be placed on the safety of maintenance personnel and long-term maintenance costs over the service life for the proposed project rather than on constructability or initial costs. Minimizing exposure to traffic through appropriate pavement type selection and sound engineering practices should always be a high priority. The District Maintenance Engineer and Maintenance Supervisor responsible for maintaining the project after it is built should be consulted for recommendations on addressing maintainability.

618.2 Constructibility

Construction issues that influence pavement type selection include: size and complexity of the project, stage construction, lane closure requirements, traffic control and safety during construction, construction windows when the project must be completed, and other constructibility issues that have the potential of generating contract change orders.

The Project Engineer must be cognizant of the issues involved in constructing a pavement, and provide plans and specifications that both meets performance standards and requirements. The construction engineer for the area where the pavement will be built should be consulted regarding constructibility during the project development process. The recommendations given by Construction should be weighed against other recommendations and requirements for the pavement. Constructibility recommendations should be accommodated where practical, provide minimum performance requirements, safety, and maintainability. Some constructibility items that should be addressed in the project include:

- Clearance width of paving machines to barriers and hinge points.
- Access for delivery trucks and construction equipment.
- Public safety and convenience.
- Time and cost of placing multiple thin lifts of different materials as opposed to thicker lifts of a single material. (For example, sometimes it is more efficient and less costly to place one thick lift of aggregate base rather than two thin lifts of aggregate base and subbase).
- The impact of combined lifts of different materials on long-term performance or maintenance of the pavement. (For example, it may seem to be a good idea to combine layers of portland cement concrete and lean concrete base into a single layer to make it easier to construct, but combining these layers has a negative impact on the pavement performance and will lead to untimely failure).
- Time and cost of using multiple types of hot mix asphalt on a project in an area away from commercial hot mix asphalt sources.

Topic 619 - Life-Cycle Cost Analysis

619.1 Life-Cycle Cost Analysis

Life-cycle cost analysis (LCCA) is a useful tool for comparing the value of alternative pavement structures and strategies. LCCA is an economic analysis that compares initial cost, future cost, and user delay cost of different pavement alternatives. LCCA is an integral part of the decision making process for selecting pavement type and design strategy. It can be used to compare life-cycle cost for:

- Different pavement types (rigid, flexible, composite).
- Different rehabilitation strategies.

- Different pavement design lives. (5 vs. 10, 10 vs. 20, 20 vs. 40, etc).

LCCA comparisons must be made between properly engineered, viable pavement structures that would be approved for construction if selected. The alternatives being evaluated should also have identical improvements. For example, comparing 10-year rehabilitation vs. 20-year rehabilitation or flexible pavement new construction vs. rigid pavement new construction, provide an identical improvement. Conversely, comparing pavement rehabilitation to new construction, or pavement overlay to pavement widening are not identical improvements.

LCCA can also be useful to determine the value of combining several projects into a single project. For example, combining a pavement rehabilitation project with a pavement widening project may reduce overall user delay and construction cost. In such case, LCCA can help determine if combining projects can reduce overall user delay and construction cost for more efficient and cost-effective projects. LCCA could also be used to identify and measure the impacts of splitting a project into two or more projects.

LCCA must conform to the procedures and data in the Life-Cycle Cost Analysis Procedures Manual. LCCA must be completed for any project with a pavement cost component except for the following:

- Major maintenance projects.
- Minor A and Minor B projects.
- Projects using Permit Engineering Evaluation Reports (PEER).
- Maintenance pullouts.
- Landscape.

For the above exempted projects, the Project Manager and the Project Development Team (PDT) will determine on a case-by-case basis if and how a life-cycle cost analysis should be performed and documented. Information on how to document life-cycle costs can be found in the Department’s Project Development Procedures Manual, Chapter 8.

CHAPTER 620
RIGID PAVEMENT

Topic 621 - Types of Rigid Pavements

Index 621.1 Jointed Plain Concrete Pavement (JPCP)

JPCP is the most common type of rigid pavement used by the Department. JPCP is engineered with longitudinal and transverse joints to control where cracking occurs in the slabs (see Figure 621.1). JPCPs do not contain steel reinforcement, other than tie bars and dowel bars (see Index 622.4 for tie bars and dowel bars). Additional guidance for JPCP can be found in the "Jointed Plain Concrete Pavement Design Guide" on the Department Pavement website.

621.2 Continuously Reinforced Concrete Pavement (CRCP)

Although the Department has used CRCP on a limited basis in the past, CRCP is still a relatively new concept to California. For this reason, the Department has decided not to use CRCP for TIs less than 11.5 or in High Mountain and High Desert climate regions. Since CRCP uses reinforcing steel rather than weakened plane joints for crack control, saw cutting of transverse joints is not required for CRCP. Longitudinal joints are still used. Transverse random cracks are expected in the slab, usually at 1.0 m to 1.5 m intervals (see Figure 621.1). The continuous reinforcement in the pavement holds the cracks tightly together. CRCP typically costs more initially than JPCP due to the added cost of the reinforcement. However, CRCP is typically more cost-effective over the life of the pavement on high volume routes due to improved long-term performance and reduced maintenance. Because there are no sawn transverse joints, properly built CRCP should have better ride quality and less maintenance than JPCP. Additional CRCP guidance are under development and when completed will be posted in the "Continuously Reinforced Concrete Pavement Design Guide" on the Department Pavement website.

621.3 Precast Panel Concrete Pavement (PPCP)

PPCPs use panels that are precast off-site instead of cast-in-place. The precast panels can be linked together with dowel bars and tie bars or can be post-tensioned after placement. PPCP offers the advantages of:

- Improved concrete mixing and curing in a precast yard
- Reduced pavement thicknesses, which is beneficial when there are profile grade restrictions such as vertical clearances
- Shorter lane closure times, which is beneficial when there are short construction windows

The primary disadvantage of PPCP is the high cost of precasting. PPCP also needs a smooth base underneath the precast panels during construction to even out the loads on the slab and avoid uneven deflection that could lead to faulting at the joints, slab settlement, and premature cracking. PPCP is currently used on an experimental basis in California, and must follow the procedures for experimental projects and special designs discussed in Topic 606.

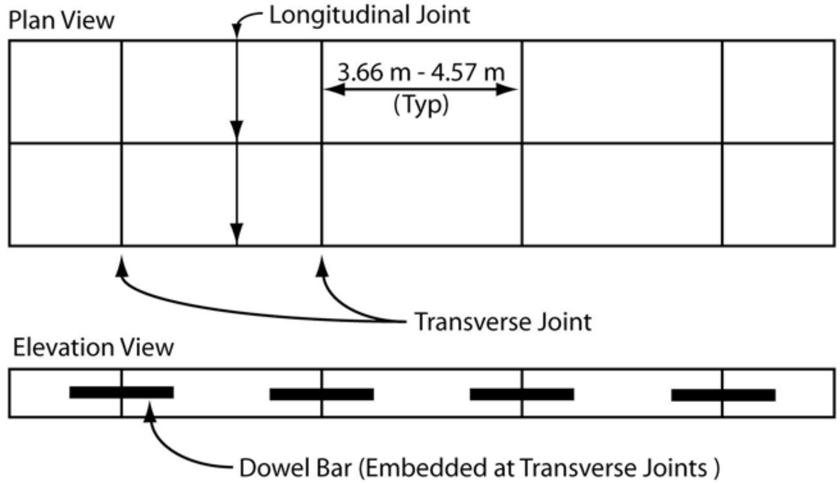
Topic 622 - Engineering Requirements

622.1 Engineering Properties

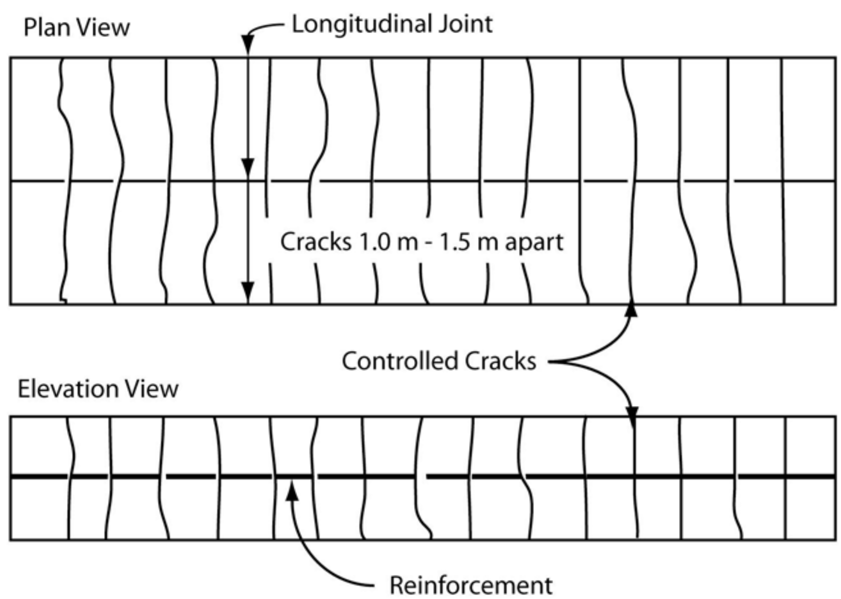
Table 622.1 shows the rigid pavement engineering properties that were used to develop the rigid pavement catalog in Index 623.1. The values are based on Department specifications and experience with materials used in California. The predominant type of concrete used in California for rigid pavement is Portland cement concrete. Other types of hydraulic cement concrete are sometimes used for special conditions such as rapid strength concrete.

Figure 621.1

Types of Rigid Pavement



Jointed Plain Concrete Pavement (JPCP)



Continuous Reinforced Concrete Pavement (CRCP)

(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 12 mm, 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	4.1 m average
Initial IRI immediately after construction	1.0 m/km (63 in/mile)
Reliability	90%
Unit weight	2400 kg/m^3
Poisson’s ratio	0.20
Coefficient of thermal expansion	10.8 x 10^{-6}/ °C
Thermal conductivity	2.16 W/m-K
Heat capacity	1.17 J/g-K
Permanent curl/warp effective temperature difference	top of slab is 5.5 °C 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)	390 kg/m^3
Water: cementitious material ratio	0.42
PCC zero-stress temperature	38.3 °C
Ultimate shrinkage at 40% relative humidity	537 microstrain
Reversible shrinkage (% of ultimate shrinkage)	50%
Time to develop ultimate shrinkage	35 days
Modulus of rupture (28 days)	4.3 MPa
Dowel bar diameter	38 mm (32 mm for rigid pavement thickness < 215 mm)

Table 622.2

Rigid Pavement Performance Factors

Factors	Values
General	
Design Life	Determined per Topic 612
Terminal IRI [(1)] at end of design life	2.54 m/km max
JPCP only	
Transverse cracking at end of design life	10% of slabs max
Longitudinal cracking at end of design life	10% of slabs max
Corner cracking at end of design life	10% of slabs max
Average joint faulting at end of design life	2.54 mm max
CRCP only	
Punchouts at end of design life	6 per kilometer max

Note:

(1) The International Roughness Index (IRI) is a nationally recognized method for measuring the smoothness of pavements.

New or reconstructed rigid pavements and lane replacements shall be tied except as noted below:

- Rigid pavement should not be tied to adjacent rigid pavement when the spacing of transverse joints of adjacent slabs is not the same.
- No more than 15 m width of rigid pavement should be tied together to preclude random longitudinal cracks from occurring due to the pavement acting as one large rigid slab. In order to maintain some load transfer across the longitudinal joint, Standard Plan P18 includes details for placing dowel bars in the longitudinal joint for this situation.

For individual slab replacements, the placement of dowel bars is determined on a project-by-project basis based on proposed design life, construction work windows, existence of dowel bars in adjacent slabs, condition of adjacent slabs, and other pertinent factors. For further information on slab replacements, see Standard Plan P8, the “Slab Replacement Guide” and supplementary “Design Tools for Slab and Lane Replacements” on the Department Pavement website.

622.5 Joint Seals

Weakened plane joints should be sealed to prevent incompressible materials from filling the joints and causing the concrete to spall. Seals also limit the entry of water that could otherwise degrade the underlying pavement layers. Various products for sealing joints are available while new ones are being developed. Each one differs in cost and service life. Recommendations on which joint seal to use should be included in the Materials Report. Typically, compression seals are preferred for new construction because of their longer performance life. Liquid sealants should be used for rehabilitation or retrofitting existing joints because they are more adaptable to surface abnormalities. For additional information on various joint seal products, consult the Pavement Technical Guidance on the Department pavement website, Standard Specifications, Standard Special Provisions, Standard Plans, or contact your District Materials Engineer.

622.6 Bond Breaker

When placing rigid pavement over a lean concrete base, it is important to avoid bonding between the two layers. Bonding can cause cracks and joints in the lean concrete base to reflect through the rigid pavement, which will lead to premature cracking. Several methods are available for preventing bonding including a liberal application of wax curing compound, or slurry seals. Application rates may be found in the Standard Specifications. For specific recommendations on how to prevent bonding between rigid pavement and lean concrete base, consult the District Materials Engineer.

622.7 Texturing

Longitudinal tining is the typical texturing for new pavements. Grooving is typically done to rehabilitate existing pavement texture or to improve surface friction. Grinding is typically done to restore a smooth riding surface on existing pavements or for individual slab replacements. Grooving or grinding are options on new pavement in lieu of longitudinal tining where there is a desire to minimize noise levels on rigid pavement.

622.8 Transitions and Anchors

Transitions and anchors are used at transverse joints to minimize deterioration or faulting of the joint where rigid pavement abuts to flexible pavement, a different rigid pavement type, or in some cases, a bridge. For JPCP, a pavement end anchor or transition should be used at transitions to flexible pavement. **For CRCP, a terminal anchor or terminal joint shall be used at all transitions to or from structure approach slabs, JPCP, PPCP, or flexible pavement.** Standard Plans include a variety of details for these transitions.

Topic 623 - Engineering Procedure for New and Reconstruction Projects

623.1 Catalog

Tables 623.1B through M contain the minimum thickness for rigid pavement surface layers, base, and subbase for all types of projects. All JPCP structures shown are doweled. The tables are categorized by subgrade soil type and climate regions. Figure 623.1 is used to determine which table to use to select the pavement structure.

The steps for selecting the appropriate rigid pavement structure are as follows:

(1) *Determine the Soil Type for the Existing Subgrade*. Soil types for existing subgrade are categorized into Types I, II, and III as shown in Table 623.1A. Soils are classified by the California R-value and Unified Soil Classification System (USCS). If a soil can be classified in more than one type in Table 623.1A, then the engineer should choose the more conservative design based on the less stable soil. Subgrade is discussed in Topic 614.

(2) *Determine Climate Region*. Find the location of the project on the Pavement Climate Map. The Pavement Climate Map is discussed in Topic 615.

(3) *Select the Appropriate Table (Tables 623.1B through M).* Select the table that applies to the project based on subgrade, soil type, and climate region. Use Figure 623.1 to determine which table applies to the project.

(4) *Determine Whether Pavement Has Lateral Support Along Both Longitudinal Joints*. The pavement is considered laterally supported if it is tied to an adjacent lane, has tied rigid shoulders, or has a widened slab. If lateral support is provided along only one longitudinal joint, then the pavement is considered to have no lateral support. As shown in Tables 623.1B through M, pavement thicknesses are reduced slightly for slabs engineered with lateral support along both longitudinal joints.

(5) *Select Pavement Structure*. Using the Traffic Index provided or calculated from the traffic projections, select the desired pavement structure from the list of alternatives provided.

Note that although the pavement structures listed for each traffic index are considered to be acceptable for the climate, soil conditions, and design life desired, they should not be considered as equal designs. Some designs will perform better than others, have lower maintenance/repair costs, and/or lower construction life-cycle costs. Sound engineering judgment should be used in selecting the option that is most effective for the location. For these reasons, the rigid pavement structures in these tables cannot be used as substitutes for the pavement structures recommended in approved Materials Reports or shown in approved contract plans.

Table 623.1A

Relationship Between Subgrade Type[1]

Subgrade Type[2]	California R-value (R)	Unified Soil Classification System (USCS)
I	R > 40	SC, SP, SM, SW, GC, GP, GM, GW
II	$10 \leq R \leq 40$	CH (PI ≤ 12), CL, MH, ML
III	R < 10	CH (PI >12)

Notes:

(1) See Topic 614 for further discussion on subgrade and USCS.

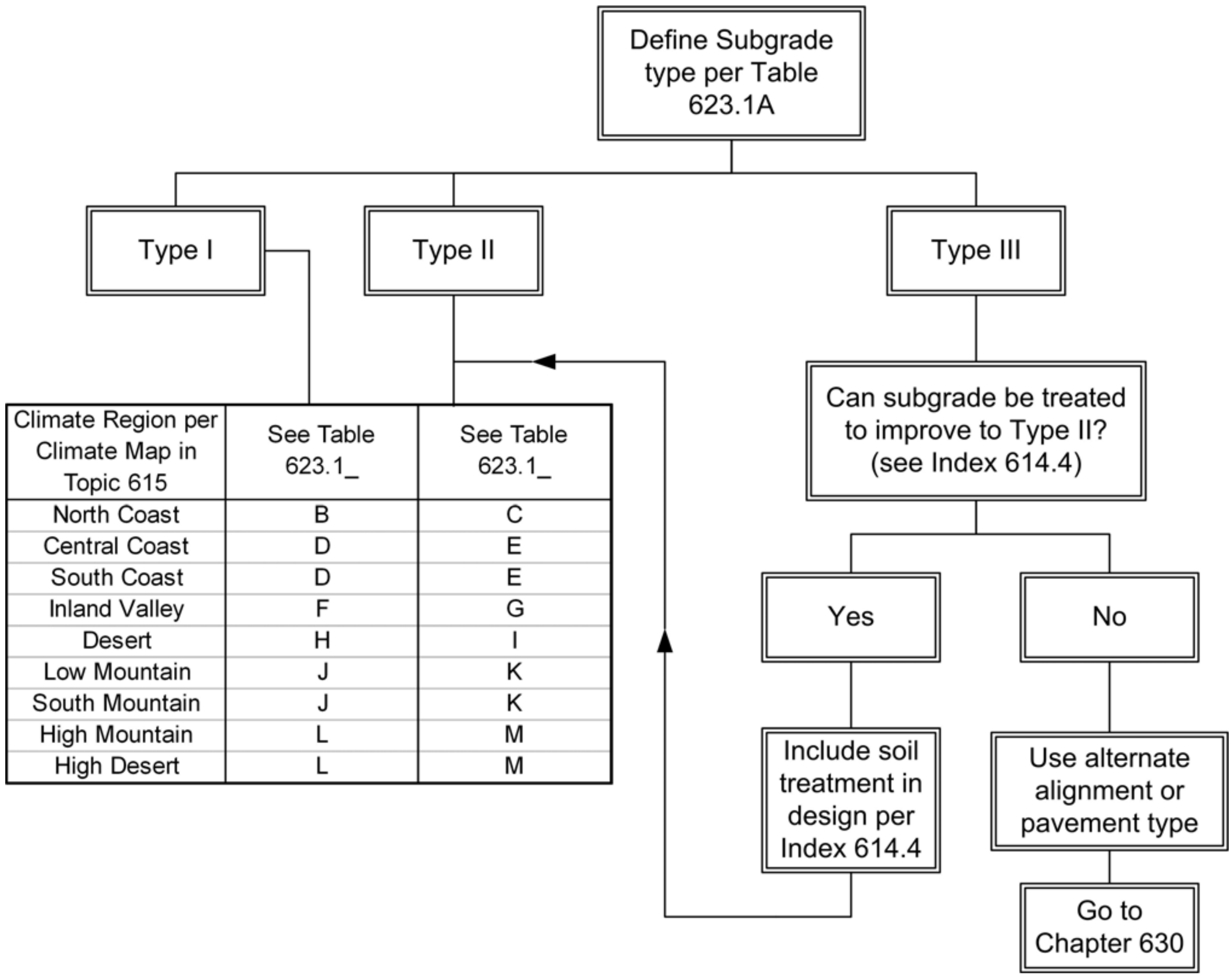
(2) Choose more conservative soil type (i.e., use soil with a lower R-value or USCS) if native soil can be classified by more than one type.

Legend

PI = Plasticity Index

Figure 623.1

Rigid Pavement Catalog Decision Tree



Climate Region per Climate Map in Topic 615	See Table 623.1_	See Table 623.1_
North Coast	B	C
Central Coast	D	E
South Coast	D	E
Inland Valley	F	G
Desert	H	I
Low Mountain	J	K
South Mountain	J	K
High Mountain	L	M
High Desert	L	M

Table 623.1B
Rigid Pavement Catalog (North Coast, Type I Subgrade Soil)(1), (2), (3), (4)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (mm)				Without Lateral Support (mm)			
< 9	210 JPCP 105 LCB	210 JPCP 105 HMA-A	210 JPCP 150 AB	210 JPCP 105 ATPB 105 AB	210 JPCP 105 LCB	210 JPCP 105 HMA-A	210 JPCP 150 AB	210 JPCP 105 ATPB 105 AB
9.5 to 10	210 JPCP 120 LCB	210 JPCP 120 HMA-A	225 JPCP 180 AB	225 JPCP 105 ATPB 120 AB	210 JPCP 120 LCB	210 JPCP 120 HMA-A	225 JPCP 180 AB	225 JPCP 105 ATPB 120 AB
10.5 to 11	210 JPCP 120 LCB	210 JPCP 120 HMA-A	210 JPCP 210 AB		225 JPCP 120 LCB	225 JPCP 120 HMA-A	225 JPCP 210 AB	
11.5 to 12	225 JPCP 120 LCB	225 JPCP 120 HMA-A	225 CRCP 120 HMA-A		240 JPCP 120 LCB	240 JPCP 120 HMA-A	240 CRCP 120 HMA-A	
12.5 to 13	240 JPCP 150 LCB	240 JPCP 150 HMA-A	225 CRCP 150 HMA-A		255 JPCP 150 LCB	255 JPCP 150 HMA-A	240 CRCP 150 HMA-A	
13.5 to 14	240 JPCP 150 LCB	240 JPCP 150 HMA-A	225 CRCP 150 HMA-A		270 JPCP 150 LCB	255 JPCP 150 HMA-A	240 CRCP 150 HMA-A	
14.5 to 15	255 JPCP 150 LCB	255 JPCP 150 HMA-A	240 CRCP 150 HMA-A		285 JPCP 150 LCB	285 JPCP 150 HMA-A	255 CRCP 150 HMA-A	
15.5 to 16	270 JPCP 150 LCB	270 JPCP 150 HMA-A	255 CRCP 150 HMA-A		300 JPCP 150 LCB	300 JPCP 150 HMA-A	270 CRCP 150 HMA-A	
16.5 to 17	285 JPCP 150 LCB	285 JPCP 150 HMA-A	255 CRCP 150 HMA-A		315 JPCP 150 LCB	315 JPCP 150 HMA-A	285 CRCP 150 HMA-A	
> 17	300 JPCP 150 LCB	300 JPCP 150 HMA-A	270 CRCP 150 HMA-A		330 JPCP 150 LCB	330JPCP 150 HMA-A	300 CRCP 150 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 10 mm sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Table 623.1C
Rigid Pavement Catalog (North Coast, Type II Subgrade Soil) (1), (2), (3), (4)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (mm)				Without Lateral Support (mm)			
≤ 9	210 JPCP 105 LCB 150 AS	210 JPCP 105 HMA-A 150 AS	210 JPCP 300 AB	210 JPCP 105 ATPB 240 AB	210 JPCP 105 LCB 150 AS	210 JPCP 105 HMA-A 150 AS	210 JPCP 300 AB	210 JPCP 105 ATPB 240 AB
9.5 to 10	210 JPCP 105 LCB 150 AS	210 JPCP 105 HMA-A 150 AS	225 JPCP 300 AB	225 JPCP 105 ATPB 240 AB	210 JPCP 105 LCB 150 AS	210 JPCP 105 HMA-A 150 AS	225 JPCP 300 AB	225 JPCP 105 ATPB 240 AB
10.5 to 11	210 JPCP 120 LCB 180 AS	210 JPCP 120 HMA-A 180 AS	210 JPCP 390 AB		225 JPCP 120 LCB 180 AS	225 JPCP 120 HMA-A 180 AS	225 JPCP 390 AB	
11.5 to 12	225 JPCP 120 LCB 180 AS	225 JPCP 120 HMA-A 180 AS	225 CRCP 120 HMA-A 180 AS		240 JPCP 120 LCB 180 AS	240 JPCP 120 HMA-A 180 AS	240 CRCP 120 HMA-A 180 AS	
12.5 to 13	240 JPCP 150 LCB 210 AS	240 JPCP 150 HMA-A 210 AS	225 CRCP 150 HMA-A 210 AS		255 JPCP 150 LCB 210 AS	255 JPCP 150 HMA-A 210 AS	240 CRCP 150 HMA-A 210 AS	
13.5 to 14	240 JPCP 150 LCB 210 AS	240 JPCP 150 HMA-A 210 AS	225 CRCP 150 HMA-A 210 AS		270 JPCP 150 LCB 210 AS	255 JPCP 150 HMA-A 210 AS	240 CRCP 150 HMA-A 210 AS	
14.5 to 15	255 JPCP 150 LCB 210 AS	255 JPCP 150 HMA-A 210 AS	240 CRCP 150 HMA-A 210 AS		285 JPCP 150 LCB 210 AS	285 JPCP 150 HMA-A 210 AS	255 CRCP 150 HMA-A 210 AS	
15.5 to 16	270 JPCP 150 LCB 210 AS	270 JPCP 150 HMA-A 210 AS	255 CRCP 150 HMA-A 210 AS		300 JPCP 150 LCB 210 AS	300 JPCP 150 HMA-A 210 AS	270 CRCP 150 HMA-A 210 AS	
16.5 to 17	285 JPCP 150 LCB 210 AS	285 JPCP 150 HMA-A 210 AS	255 CRCP 150 HMA-A 210 AS		315 JPCP 150 LCB 210 AS	315 JPCP 150 HMA-A 210 AS	285 CRCP 150 HMA-A 210 AS	
> 17	300 JPCP 150 LCB 210 AS	300 JPCP 150 HMA-A 210 AS	270 CRCP 150 HMA-A 210 AS		330 JPCP 150 LCB 210 AS	330 JPCP 150 HMA-A 210 AS	300 CRCP 150 HMA-A 210 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 10 mm sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

Legend:

JPCP =	Jointed Plain Concrete Pavement	ATPB =	Asphalt Treated Permeable Base
CRCP =	Continuously Reinforced Concrete Pavement	AB =	Class 2 Aggregate Base
LCB =	Lean Concrete Base	AS =	Class 2 Aggregate Subbase
HMA-A =	Hot Mix Asphalt (Type A)	TI =	Traffic Index

Table 623.1D
Rigid Pavement Catalog
(South Coast/Central Coast, Type I Subgrade Soil) (1), (2), (3), (4)

TI	Rigid Pavement Structural Depth							
	With Lateral Support (mm)				Without Lateral Support (mm)			
< 9	210 JPCP 105 LCB	210 JPCP 105 HMA-A	210 JPCP 150 AB	210 JPCP 105 ATPB 105 AB	210 JPCP 105 LCB	210 JPCP 105 HMA-A	225 JPCP 150 AB	225 JPCP 105 ATPB 105 AB
9.5 to 10	210 JPCP 120 LCB	210 JPCP 120 HMA-A	225 JPCP 180 AB	225 JPCP 105 ATPB 120 AB	225 JPCP 120 LCB	225 JPCP 120 HMA-A	240 JPCP 180 AB	240 JPCP 105 ATPB 120 AB
10.5 to 11	225 JPCP 120 LCB	225 JPCP 120 HMA-A	240 JPCP 210 AB		240 JPCP 120 LCB	240 JPCP 120 HMA-A	255 JPCP 210 AB	
11.5 to 12	240 JPCP 120 LCB	240 JPCP 120 HMA-A	240 CRCP 120 HMA-A		255 JPCP 120 LCB	255 JPCP 120 HMA-A	240 CRCP 120 HMA-A	
12.5 to 13	255 JPCP 150 LCB	255 JPCP 150 HMA-A	240 CRCP 150 HMA-A		270 JPCP 150 LCB	270 JPCP 150 HMA-A	255 CRCP 150 HMA-A	
13.5 to 14	255 JPCP 150 LCB	255 JPCP 150 HMA-A	240 CRCP 150 HMA-A		285 JPCP 150 LCB	285 JPCP 150 HMA-A	270 CRCP 150 HMA-A	
14.5 to 15	270 JPCP 150 LCB	270 JPCP 150 HMA-A	255 CRCP 150 HMA-A		300 JPCP 150 LCB	300 JPCP 150 HMA-A	285 CRCP 150 HMA-A	
15.5 to 16	285 JPCP 150 LCB	270 JPCP 150 HMA-A	255 CRCP 150 HMA-A		315 JPCP 150 LCB	315 JPCP 150 HMA-A	285 CRCP 150 HMA-A	
16.5 to 17	300 JPCP 150 LCB	285 JPCP 150 HMA-A	270 CRCP 150 HMA-A		330 JPCP 150 LCB	330 JPCP 150 HMA-A	300 CRCP 150 HMA-A	
> 17	315 JPCP 150 LCB	315 JPCP 150 HMA-A	285 CRCP 150 HMA-A		345 JPCP 150 LCB	345 JPCP 150 HMA-A	300 CRCP 150 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 10 mm sacrificial wearing course for future grinding of JPCP/CRCP.
(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.
(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Table 623.1E
Rigid Pavement Catalog
(South Coast/Central Coast, Type II Subgrade Soil) (1), (2), (3), (4)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (mm)				Without Lateral Support (mm)			
< 9	210 JPCP 105 LCB 150 AS	210 JPCP 105 HMA-A 150 AS	210 JPCP 300 AB	210 JPCP 105 ATPB 240 AB	210 JPCP 105 LCB 150 AS	210 JPCP 105 HMA-A 150 AS	225 JPCP 300 AB	225 JPCP 105 ATPB 240 AB
9.5 to 10	210 JPCP 105 LCB 150 AS	210 JPCP 105 HMA-A 150 AS	225 JPCP 300 AB	225 JPCP 105 ATPB 240 AB	225 JPCP 105 LCB 150 AS	225 JPCP 105 HMA-A 150 AS	240 JPCP 300 AB	240 JPCP 105 ATPB 240 AB
10.5 to 11	225 JPCP 120 LCB 180 AS	225 JPCP 120 HMA-A 180 AS	240 JPCP 390 AB		240 JPCP 120 LCB 180 AS	240 JPCP 120 HMA-A 180 AS	255 JPCP 390 AB	
11.5 to 12	240 JPCP 120 LCB 180 AS	240 JPCP 120 HMA-A 180 AS	240 CRCP 120 HMA-A 180 AS		255 JPCP 120 LCB 180 AS	255 JPCP 120 HMA-A 180 AS	240 CRCP 120 HMA-A 180 AS	
12.5 to 13	255 JPCP 150 LCB 210 AS	255 JPCP 150 HMA-A 210 AS	240 CRCP 150 HMA-A 210 AS		270 JPCP 150 LCB 210 AS	270 JPCP 150 HMA-A 210 AS	255 CRCP 150 HMA-A 210 AS	
13.5 to 14	255 JPCP 150 LCB 210 AS	255 JPCP 150 HMA-A 210 AS	240 CRCP 150 HMA-A 210 AS		285 JPCP 150 LCB 210 AS	285 JPCP 150 HMA-A 210 AS	270 CRCP 150 HMA-A 210 AS	
14.5 to 15	270 JPCP 150 LCB 210 AS	270 JPCP 150 HMA-A 210 AS	255 CRCP 150 HMA-A 210 AS		300 JPCP 150 LCB 210 AS	300 JPCP 150 HMA-A 210 AS	285 CRCP 150 HMA-A 210 AS	
15.5 to 16	285 JPCP 150 LCB 210 AS	270 JPCP 150 HMA-A 210 AS	255 CRCP 150 HMA-A 210 AS		315 JPCP 150 LCB 210 AS	315 JPCP 150 HMA-A 210 AS	285 CRCP 150 HMA-A 210 AS	
16.5 to 17	300 JPCP 150 LCB 210 AS	285 JPCP 150 HMA-A 210 AS	270 CRCP 150 HMA-A 210 AS		330 JPCP 150 LCB 210 AS	330 JPCP 150 HMA-A 210 AS	300 CRCP 150 HMA-A 210 AS	
> 17	315 JPCP 150 LCB 210 AS	315 JPCP 150 HMA-A 210 AS	285 CRCP 150 HMA-A 210 AS		345 JPCP 150 LCB 210 AS	345 JPCP 150 HMA-A 210 AS	300 CRCP 150 HMA-A 210 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 10 mm sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
AS = Class 2 Aggregate Subbase
TI = Traffic Index

Table 623.1F
Rigid Pavement Catalog (Inland Valley, Type I Subgrade Soil) (1), (2), (3), (4)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (mm)				Without Lateral Support (mm)			
< 9	210 JPCP 105 LCB	210 JPCP 105 HMA-A	225 JPCP 150 AB	210 JPCP 105 ATPB 105 AB	225 JPCP 105 LCB	225 JPCP 105 HMA-A	240 JPCP 150 AB	225 JPCP 105 ATPB 105 AB
9.5 to 10	210 JPCP 120 LCB	210 JPCP 120 HMA-A	240 JPCP 180 AB	225 JPCP 105 ATPB 120 AB	240 JPCP 120 LCB	255 JPCP 120 HMA-A	270 JPCP 180 AB	255 JPCP 105 ATPB 120 AB
10.5 to 11	225 JPCP 120 LCB	225 JPCP 120 HMA-A	255 JPCP 210 AB		255 JPCP 120 LCB	270 JPCP 120 HMA-A	285 JPCP 210 AB	
11.5 to 12	255 JPCP 120 LCB	255 JPCP 120 HMA-A	240 CRCP 120 HMA-A		285 JPCP 120 LCB	285 JPCP 120 HMA-A	255 CRCP 120 HMA-A	
12.5 to 13	255 JPCP 150 LCB	270 JPCP 150 HMA-A	240 CRCP 150 HMA-A		300 JPCP 150 LCB	300 JPCP 150 HMA-A	270 CRCP 150 HMA-A	
13.5 to 14	285 JPCP 150 LCB	285 JPCP 150 HMA-A	255 CRCP 150 HMA-A		315 JPCP 150 LCB	315 JPCP 150 HMA-A	285 CRCP 150 HMA-A	
14.5 to 15	300 JPCP 150 LCB	300 JPCP 150 HMA-A	270 CRCP 150 HMA-A		345 JPCP 150 LCB	345 JPCP 150 HMA-A	300 CRCP 150 HMA-A	
15.5 to 16	315 JPCP 150 LCB	315 JPCP 150 HMA-A	285 CRCP 150 HMA-A		360 JPCP 150 LCB	360 JPCP 150 HMA-A	315 CRCP 150 HMA-A	
16.5 to 17	330 JPCP 150 LCB	330 JPCP 150 HMA-A	285 CRCP 150 HMA-A		375 JPCP 150 LCB	375 JPCP 150 HMA-A	330 CRCP 150 HMA-A	
> 17	345 JPCP 150 LCB	345 JPCP 150 HMA-A	300 CRCP 150 HMA-A		390 JPCP 150 LCB	390 JPCP 150 HMA-A	330 CRCP 150 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 10 mm sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Table 623.1G
Rigid Pavement Catalog (Inland Valley, Type II Subgrade Soil) (1), (2), (3), (4)

TI	Rigid Pavement Structural Depth							
	With Lateral Support (mm)				Without Lateral Support (mm)			
< 9	210 JPCP 105 LCB 150 AS	210 JPCP 105 HMA-A 150 AS	225 JPCP 300 AB	210 JPCP 105 ATPB 240 AB	225 JPCP 105 LCB 150 AS	225 JPCP 105 HMA-A 150 AS	240 JPCP 300 AB	225 JPCP 105 ATPB 240 AB
9.5 to 10	210 JPCP 105 LCB 150 AS	210 JPCP 105 HMA-A 150 AS	240 JPCP 300 AB	225 JPCP 105 ATPB 240 AB	240 JPCP 105 LCB 150 AS	255 JPCP 105 HMA-A 150 AS	270 JPCP 300 AB	255 JPCP 105 ATPB 240 AB
10.5 to 11	225 JPCP 120 LCB 180 AS	225 JPCP 120 HMA-A 180 AS	255 JPCP 390 AB		255 JPCP 120 LCB 180 AS	270 JPCP 120 HMA-A 180 AS	285 JPCP 390 AB	
11.5 to 12	255 JPCP 120 LCB 180 AS	255 JPCP 120 HMA-A 180 AS	240 CRCP 120 HMA-A 180 AS		285 JPCP 120 LCB 180 AS	285 JPCP 120 HMA-A 180 AS	255 CRCP 120 HMA-A 180 AS	
12.5 to 13	255 JPCP 150 LCB 210 AS	270 JPCP 150 HMA-A 210 AS	240 CRCP 150 HMA-A 210 AS		300 JPCP 150 LCB 210 AS	300 JPCP 150 HMA-A 210 AS	270 CRCP 150 HMA-A 210 AS	
13.5 to 14	285 JPCP 150 LCB 210 AS	285 JPCP 150 HMA-A 210 AS	255 CRCP 150 HMA-A 210 AS		315 JPCP 150 LCB 210 AS	315 JPCP 150 HMA-A 210 AS	285 CRCP 150 HMA-A 210 AS	
14.5 to 15	300 JPCP 150 LCB 210 AS	300 JPCP 150 HMA-A 210 AS	270 CRCP 150 HMA-A 210 AS		345 JPCP 150 LCB 210 AS	345 JPCP 150 HMA-A 210 AS	300 CRCP 150 HMA-A 210 AS	
15.5 to 16	315 JPCP 150 LCB 210 AS	315 JPCP 150 HMA-A 210 AS	285 CRCP 150 HMA-A 210 AS		360 JPCP 150 LCB 210 AS	360 JPCP 150 HMA-A 210 AS	315 CRCP 150 HMA-A 210 AS	
16.5 to 17	330 JPCP 150 LCB 210 AS	330 JPCP 150 HMA-A 210 AS	285 CRCP 150 HMA-A 210 AS		375 JPCP 150 LCB 210 AS	375 JPCP 150 HMA-A 210 AS	330 CRCP 150 HMA-A 210 AS	
> 17	345 JPCP 150 LCB 210 AS	345 JPCP 150 HMA-A 210 AS	300 CRCP 150 HMA-A 210 AS		390 JPCP 150 LCB 210 AS	390 JPCP 150 HMA-A 210 AS	330 CRCP 150 HMA-A 210 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 10 mm sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

Legend:

JPCP =	Jointed Plain Concrete Pavement	ATPB =	Asphalt Treated Permeable Base
CRCP =	Continuously Reinforced Concrete Pavement	AB =	Class 2 Aggregate Base
LCB =	Lean Concrete Base	AS =	Class 2 Aggregate Subbase
HMA-A =	Hot Mix Asphalt (Type A)	TI =	Traffic Index

Table 623.1H
Rigid Pavement Catalog (Desert, Type I Subgrade Soil) (1), (2), (3), (4)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (mm)				Without Lateral Support (mm)			
< 9	210 JPCP 105 LCB	210 JPCP 105 HMA-A	225 JPCP 150 AB	210 JPCP 105 ATPB 120 AB	225 JPCP 105 LCB	225 JPCP 105 HMA-A	240 JPCP 150 AB	225 JPCP 105 ATPB 120 AB
9.5 to 10	225 JPCP 120 LCB	225 JPCP 120 HMA-A	240 JPCP 180 AB	240 JPCP 105 ATPB 120 AB	240 JPCP 120 LCB	255 JPCP 120 HMA-A	270 JPCP 180 AB	255 JPCP 105 ATPB 120 AB
10.5 to 11	240 JPCP 120 LCB	240 JPCP 120 HMA-A	255 JPCP 210 AB		255 JPCP 120 LCB	270 JPCP 120 HMA-A	285 JPCP 210 AB	
11.5 to 12	255 JPCP 120 LCB	255 JPCP 120 HMA-A	240 CRCP 120 HMA-A		270 JPCP 120 LCB	285 JPCP 120 HMA-A	255 CRCP 120 HMA-A	
12.5 to 13	285 JPCP 150 LCB	285 JPCP 150 HMA-A	255 CRCP 150 HMA-A		315 JPCP 150 LCB	315 JPCP 150 HMA-A	285 CRCP 150 HMA-A	
13.5 to 14	300 JPCP 150 LCB	300 JPCP 150 HMA-A	270 CRCP 150 HMA-A		345 JPCP 150 LCB	345 JPCP 150 HMA-A	315 CRCP 150 HMA-A	
14.5 to 15	315 JPCP 150 LCB	315 JPCP 150 HMA-A	285 CRCP 150 HMA-A		360 JPCP 150 LCB	360 JPCP 150 HMA-A	330 CRCP 150 HMA-A	
15.5 to 16	330 JPCP 150 LCB	330 JPCP 150 HMA-A	300 CRCP 150 HMA-A		375 JPCP 150 LCB	375 JPCP 150 HMA-A	330 CRCP 150 HMA-A	
16.5 to 17	345 JPCP 150 LCB	345 JPCP 150 HMA-A	315 CRCP 150 HMA-A		390 JPCP 150 LCB	390 JPCP 150 HMA-A	330 CRCP 150 HMA-A	
> 17	360 JPCP 150 LCB	360 JPCP 150 HMA-A	330 CRCP 150 HMA-A		390 JPCP 150 LCB	390 JPCP 150 HMA-A	330 CRCP 150 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 10 mm sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Table 623.1I
Rigid Pavement Catalog (Desert, Type II Subgrade Soil) (1), (2), (3), (4)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (mm)				Without Lateral Support (mm)			
< 9	210 JPCP 105 LCB 150 AS	210 JPCP 105 HMA-A 150 AS	225 JPCP 300 AB	210 JPCP 105 ATPB 240 AB	225 JPCP 105 LCB 180 AS	225 JPCP 105 HMA-A 180 AS	240 JPCP 300 AB	225 JPCP 105 ATPB 240 AB
9.5 to 10	225 JPCP 120 LCB 150 AS	225 JPCP 120 HMA-A 150 AS	240 JPCP 300 AB	240 JPCP 105 ATPB 240 AB	240 JPCP 120 LCB 180 AS	255 JPCP 120 HMA-A 180 AS	270 JPCP 300 AB	255 JPCP 105 ATPB 240 AB
10.5 to 11	240 JPCP 120 LCB 180 AS	240 JPCP 120 HMA-A 180 AS	255 JPCP 390 AB		255 JPCP 120 LCB 180 AS	270 JPCP 120 HMA-A 180 AS	285 JPCP 390 AB	
11.5 to 12	255 JPCP 120 LCB 180 AS	255 JPCP 120 HMA-A 180 AS	240 CRCP 120 HMA-A 180 AS		270 JPCP 120 LCB 180 AS	285 JPCP 120 HMA-A 180 AS	255 CRCP 120 HMA-A 180 AS	
12.5 to 13	285 JPCP 150 LCB 210 AS	285 JPCP 150 HMA-A 210 AS	255 CRCP 150 HMA-A 210 AS		315 JPCP 150 LCB 210 AS	315 JPCP 150 HMA-A 210 AS	285 CRCP 150 HMA-A 210 AS	
13.5 to 14	300 JPCP 150 LCB 210 AS	300 JPCP 150 HMA-A 210 AS	270 CRCP 150 HMA-A 210 AS		345 JPCP 150 LCB 210 AS	345 JPCP 150 HMA-A 210 AS	315 CRCP 150 HMA-A 210 AS	
14.5 to 15	315 JPCP 150 LCB 210 AS	315 JPCP 150 HMA-A 210 AS	285 CRCP 150 HMA-A 210 AS		360 JPCP 150 LCB 210 AS	360 JPCP 150 HMA-A 210 AS	330 CRCP 150 HMA-A 210 AS	
15.5 to 16	330 JPCP 150 LCB 210 AS	330 JPCP 150 HMA-A 210 AS	300 CRCP 150 HMA-A 210 AS		375 JPCP 150 LCB 210 AS	375 JPCP 150 HMA-A 210 AS	330 CRCP 150 HMA-A 210 AS	
16.5 to 17	345 JPCP 150 LCB 210 AS	345 JPCP 150 HMA-A 210 AS	315 CRCP 150 HMA-A 210 AS		390 JPCP 150 LCB 210 AS	390 JPCP 150 HMA-A 210 AS	330 CRCP 150 HMA-A 210 AS	
> 17	360 JPCP 150 LCB 210 AS	360 JPCP 150 HMA-A 210 AS	330 CRCP 150 HMA-A 210 AS		390 JPCP 150 LCB 210 AS	390 JPCP 150 HMA-A 210 AS	330 CRCP 150 HMA-A 210 AS	

Notes:

(1) Thicknesses shown are for doweled JPCP only. Not valid for nondoweled JPCP.

(2) Includes 10 mm sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

Legend:

JPCP =	Jointed Plain Concrete Pavement	ATPB =	Asphalt Treated Permeable Base
CRCP =	Continuously Reinforced Concrete Pavement	AB =	Class 2 Aggregate Base
LCB =	Lean Concrete Base	AS =	Class 2 Aggregate Subbase
HMA-A =	Hot Mix Asphalt (Type A)	TI =	Traffic Index

Table 623.1J
Rigid Pavement Catalog
(Low Mountain/South Mountain, Type I Subgrade Soil) (1), (2), (3), (4)

TI	Rigid Pavement Structural Depth: With Lateral Support (mm)				Without Lateral Support (mm)			
< 9	210 JPCP 105 LCB	210 JPCP 105 HMA-A	225 JPCP 150 AB	210 JPCP 105 ATPB 120 AB	225 JPCP 105 LCB	225 JPCP 105 HMA-A	225 JPCP 150 AB	225 JPCP 105 ATPB 120 AB
9.5 to 10	210 JPCP 120 LCB	210 JPCP 120 HMA-A	225 JPCP 180 AB	225 JPCP 105 ATPB 120 AB	240 JPCP 120 LCB	240 JPCP 120 HMA-A	255 JPCP 180 AB	240 JPCP 105 ATPB 120 AB
10.5 to 11	225 JPCP 120 LCB	225 JPCP 120 HMA-A	240 JPCP 210 AB		255 JPCP 120 LCB	255 JPCP 120 HMA-A	270 JPCP 210 AB	
11.5 to 12	240 JPCP 120 LCB	255 JPCP 120 HMA-A	240 CRCP 120 HMA-A		270 JPCP 120 LCB	285 JPCP 120 HMA-A	255 CRCP 120 HMA-A	
12.5 to 13	270 JPCP 150 LCB	285 JPCP 150 HMA-A	255 CRCP 150 HMA-A		300 JPCP 150 LCB	315 JPCP 150 HMA-A	270 CRCP 150 HMA-A	
13.5 to 14	285 JPCP 150 LCB	300 JPCP 150 HMA-A	255 CRCP 150 HMA-A		315 JPCP 150 LCB	330 JPCP 150 HMA-A	285 CRCP 150 HMA-A	
14.5 to 15	300 JPCP 150 LCB	315 JPCP 150 HMA-A	270 CRCP 150 HMA-A		345 JPCP 150 LCB	360 JPCP 150 HMA-A	315 CRCP 150 HMA-A	
15.5 to 16	315 JPCP 150 LCB	330 JPCP 150 HMA-A	285 CRCP 150 HMA-A		360 JPCP 150 LCB	375 JPCP 150 HMA-A	330 CRCP 150 HMA-A	
16.5 to 17	330 JPCP 150 LCB	345 JPCP 150 HMA-A	300 CRCP 150 HMA-A		375 JPCP 150 LCB	390 JPCP 150 HMA-A	330 CRCP 150 HMA-A	
> 17	345 JPCP 150 LCB	360 JPCP 150 HMA-A	300 CRCP 150 HMA-A		390 JPCP 150 LCB	405 JPCP 150 HMA-A	330 CRCP 150 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 10 mm sacrificial wearing course for future grinding of JPCP/CRCP.
(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.
(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

Legend:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index

Table 623.1K
Rigid Pavement Catalog
(Low Mountain/South Mountain, Type II Subgrade Soil) (1), (2), (3), (4)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (mm)				Without Lateral Support (mm)			
< 9	210 JPCP 105 LCB 150 AS	210 JPCP 105 HMA-A 150 AS	225 JPCP 300 AB	210 JPCP 105 ATPB 240 AB	225 JPCP 105 LCB 150 AS	225 JPCP 105 HMA-A 150 AS	225 JPCP 300 AB	225 JPCP 105 ATPB 240 AB
9.5 to 10	210 JPCP 120 LCB 150 AS	210 JPCP 120 HMA-A 150 AS	225 JPCP 300 AB	225 JPCP 105 ATPB 240 AB	240 JPCP 120 LCB 150 AS	240 JPCP 120 HMA-A 150 AS	255 JPCP 300 AB	240 JPCP 105 ATPB 240 AB
10.5 to 11	225 JPCP 120 LCB 180 AS	225 JPCP 120 HMA-A 180 AS	240 JPCP 390 AB		255 JPCP 120 LCB 180 AS	255 JPCP 120 HMA-A 180 AS	270 JPCP 390 AB	
11.5 to 12	240 JPCP 120 LCB 180 AS	255 JPCP 120 HMA-A 180 AS	240 CRCP 120 HMA-A 180 AS		270 JPCP 120 LCB 180 AS	285 JPCP 120 HMA-A 180 AS	255 CRCP 120 HMA-A 180 AS	
12.5 to 13	270 JPCP 150 LCB 210 AS	285 JPCP 150 HMA-A 210 AS	255 CRCP 150 HMA-A 210 AS		300 JPCP 150 LCB 210 AS	315 JPCP 150 HMA-A 210 AS	270 CRCP 150 HMA-A 210 AS	
13.5 to 14	285 JPCP 150 LCB 210 AS	300 JPCP 150 HMA-A 210 AS	255 CRCP 150 HMA-A 210 AS		315 JPCP 150 LCB 210 AS	330 JPCP 150 HMA-A 210 AS	285 CRCP 150 HMA-A 210 AS	
14.5 to 15	300 JPCP 150 LCB 210 AS	315 JPCP 150 HMA-A 210 AS	270 CRCP 150 HMA-A 210 AS		345 JPCP 150 LCB 210 AS	360 JPCP 150 HMA-A 210 AS	315 CRCP 150 HMA-A 210 AS	
15.5 to 16	315 JPCP 150 LCB 210 AS	330 JPCP 150 HMA-A 210 AS	285 CRCP 150 HMA-A 210 AS		360 JPCP 150 LCB 210 AS	375 JPCP 150 HMA-A 210 AS	330 CRCP 150 HMA-A 210 AS	
16.5 to 17	330 JPCP 150 LCB 210 AS	345 JPCP 150 HMA-A 210 AS	300 CRCP 150 HMA-A 210 AS		375 JPCP 150 LCB 210 AS	390 JPCP 150 HMA-A 210 AS	330 CRCP 150 HMA-A 210 AS	
> 17	345 JPCP 150 LCB 210 AS	360 JPCP 150 HMA-A 210 AS	300 CRCP 150 HMA-A 210 AS		390 JPCP 150 LCB 210 AS	1.35 JPCP 150 HMA-A 210 AS	330 CRCP 150 HMA-A 210 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 10 mm sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

Legend:

JPCP =	Jointed Plain Concrete Pavement	ATPB =	Asphalt Treated Permeable Base
CRCP =	Continuously Reinforced Concrete Pavement	AB =	Class 2 Aggregate Base
LCB =	Lean Concrete Base	AS =	Class 2 Aggregate Subbase
HMA-A =	Hot Mix Asphalt (Type A)	TI =	Traffic Index

Table 623.1L
Rigid Pavement Catalog
(High Mountain/High Desert, Type I Subgrade Soil) (1), (2), (3), (4)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (mm)				Without Lateral Support (mm)			
< 9	240 JPCP 105 LCB	255 JPCP 105 HMA-A	255 JPCP 150 AB	240 JPCP 105 ATPB 120 AB	255 JPCP 105 LCB	270 JPCP 105 HMA-A	270 JPCP 150 AB	270 JPCP 105 ATPB 120 AB
9.5 to 10	255 JPCP 120 LCB	255 JPCP 120 HMA-A	270 JPCP 180 AB	270 JPCP 105 ATPB 120 AB	270 JPCP 120 LCB	270 JPCP 120 HMA-A	285 JPCP 180 AB	270 JPCP 105 ATPB 120 AB
10.5 to 11	270 JPCP 120 LCB	270 JPCP 120 HMA-A	285 JPCP 210 AB		285 JPCP 120 LCB	285 JPCP 120 HMA-A	300 JPCP 210 AB	
11.5 to 12	285 JPCP 120 LCB	285 JPCP 120 HMA-A			315 JPCP 120 LCB	315 JPCP 120 HMA-A		
12.5 to 13	300 JPCP 150 LCB	315 JPCP 150 HMA-A			330 JPCP 150 LCB	345 JPCP 150 HMA-A		
13.5 to 14	315 JPCP 150 LCB	330 JPCP 150 HMA-A			345 JPCP 150 LCB	360 JPCP 150 HMA-A		
14.5 to 15	330 JPCP 150 LCB	345 JPCP 150 HMA-A			360 JPCP 150 LCB	375 JPCP 150 HMA-A		
15.5 to 16	345 JPCP 150 LCB	360 JPCP 150 HMA-A			375 JPCP 150 LCB	390 JPCP 150 HMA-A		
16.5 to 17	360 JPCP 150 LCB	375 JPCP 150 HMA-A			390 JPCP 150 LCB	405 JPCP 150 HMA-A		
> 17	375 JPCP 150 LCB	375 JPCP 150 HMA-A			405 JPCP 150 LCB	405 JPCP 150 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 45 mm 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/Low Mountain, Type II Subgrade Soil) (1), (2), (3), (4)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (mm)				Without Lateral Support (mm)			
< 9	240 JPCP 105 LCB 150 AS	255 JPCP 105 HMA-A 150 AS	255 JPCP 300 AB	240 JPCP 105 ATPB 240 AB	255 JPCP 105 LCB 150 AS	270 JPCP 105 HMA-A 150 AS	270 JPCP 300 AB	270 JPCP 105 ATPB 240 AB
9.5 to 10	255 JPCP 120 LCB 150 AS	255 JPCP 120 HMA-A 150 AS	270 JPCP 300 AB	270 JPCP 105 ATPB 240 AB	270 JPCP 120 LCB 150 AS	270 JPCP 120 HMA-A 150 AS	285 JPCP 300 AB	270 JPCP 105 ATPB 240 AB
10.5 to 11	270 JPCP 120 LCB 180 AS	270 JPCP 120 HMA-A 180 AS	285 JPCP 390 AB		285 JPCP 120 LCB 180 AS	285 JPCP 120 HMA-A 180 AS	300 JPCP 390 AB	
11.5 to 12	285 JPCP 120 LCB 180 AS	285 JPCP 120 HMA-A 180 AS			315 JPCP 120 LCB 180 AS	315 JPCP 120 HMA-A 180 AS		
12.5 to 13	300 JPCP 150 LCB 210 AS	315 JPCP 150 HMA-A 210 AS			330 JPCP 150 LCB 210 AS	345 JPCP 150 HMA-A 210 AS		
13.5 to 14	315 JPCP 150 LCB 210 AS	330 JPCP 150 HMA-A 210 AS			345 JPCP 150 LCB 210 AS	360 JPCP 150 HMA-A 210 AS		
14.5 to 15	330 JPCP 150 LCB 210 AS	345 JPCP 150 HMA-A 210 AS			360 JPCP 150 LCB 210 AS	375 JPCP 150 HMA-A 210 AS		
15.5 to 16	345 JPCP 150 LCB 210 AS	360 JPCP 150 HMA-A 210 AS			375 JPCP 150 LCB 210 AS	390 JPCP 150 HMA-A 210 AS		
16.5 to 17	360 JPCP 150 LCB 210 AS	375 JPCP 150 HMA-A 210 AS			390 JPCP 150 LCB 210 AS	405 JPCP 150 HMA-A 210 AS		
> 17	375 JPCP 150 LCB 210 AS	375 JPCP 150 HMA-A 210 AS			405 JPCP 150 LCB 210 AS	405 JPCP 150 HMA-A 210 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 45 mm 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	ATPB =	Asphalt Treated Permeable Base
CRCP =	Continuously Reinforced Concrete Pavement	AB =	Class 2 Aggregate Base
LCB =	Lean Concrete Base	AS =	Class 2 Aggregate Subbase
HMA-A =	Hot Mix Asphalt (Type A)	TI =	Traffic Index

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 30 mm 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	105 mm HMA SAMI-F or SAMI-R 30 mm HMA (LC)	60 mm RHMA-G SAMI-R 30 mm HMA (LC)
TI ≥12.0	150 mm HMA SAMI-F or SAMI-R 30 mm HMA (LC)	60 mm RHMA-G 45 mm HMA SAMI-F or SAMI-R 30 mm HMA (LC)

Notes:

(1) If the existing rigid pavement is not cracked and seated, add minimum 30 mm HMA above the SAMI layer.

Legend:

HMA = Hot Mixed Asphalt
HMA (LC) = Hot Mixed 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)

Not that there are currently no standard crack, seat, and asphalt overlay designs for pavement design lives greater than 20 years. For projects with a 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 thicknesses found in Table 625.1. Once a rigid pavement has been cracked, seated, and overlaid with asphalt 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 30 mm HMA to the minimum standard thicknesses of HMA surface course layer given in Table 625.1. Since the maximum thickness for RHMA-G is 60 mm (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 5 mm 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 5 mm 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 web site.

(4) *Selection.* The selection of the appropriate strategy should be based upon life-cycle costs,

load transfer efficiency of the joints, materials testing, ride quality, safety, maintainability, constructibility, visual inspection of pavement distress, and other factors listed in Chapter 610. The Materials Report should discuss any historical problems observed in the performance of rigid pavement constructed with aggregates found near the proposed project and subjected to similar physical and environmental conditions.

625.2 Mechanistic-Empirical Method

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

Topic 626 - Other Considerations

626.1 Traveled Way

(1) *Mainline.* No additional considerations.

(2) *Ramps and Connectors.* If tied rigid shoulders or widened slabs are used on the mainline, then the ramp or connector gore area (including ramp traveled way adjacent to the gore area) should also be constructed with rigid pavement (see Figure 626.1). This will minimize deterioration of the joint between flexible and rigid pavement. When the ramp or connector traveled way is rigid pavement, utilize the same base and thickness for the gore area as that to be used under the ramp traveled way, especially when concrete shoulders are utilized on the mainline. Note that in order to optimize constructability, any concrete pavement structure used for mainline concrete shoulders should still be perpetuated through the gore area. If the base is Treated Permeable Base (TPB) under the ramp's traveled way and shoulder, TPB should still be utilized in the ramp gore areas as well.

(3) *Ramp Termini.* Rigid pavement is sometimes placed at ramp termini instead of flexible pavement where there is projected heavy truck traffic (as defined in Index 613.5(1)(c)) to preclude pavement failure such as rutting or shoving from vehicular braking, turning movements, and oil dripping from vehicles. Once a design TI is selected for the ramp in accordance with Index 613.5, follow the requirements in Index 623.1 to engineer the rigid pavement structure for the ramp termini. The length of rigid pavement to be placed at the termini will depend on the geometric alignment of the ramp, ramp grades, and the length of queues of stopped traffic. The rigid pavement should extend to the first set of signal loops on signalized intersections. A length of 45 m should be considered the minimum on unsignalized intersections. Special care should be taken to assure skid resistance in conformance with current standard specifications in the braking area, especially where oil drippage is concentrated. End anchors or transitions should be used at flexible/rigid pavement joints. The Department pavement website has additional information and training for engineering pavement for intersections and rigid ramp termini.

626.2 Shoulder

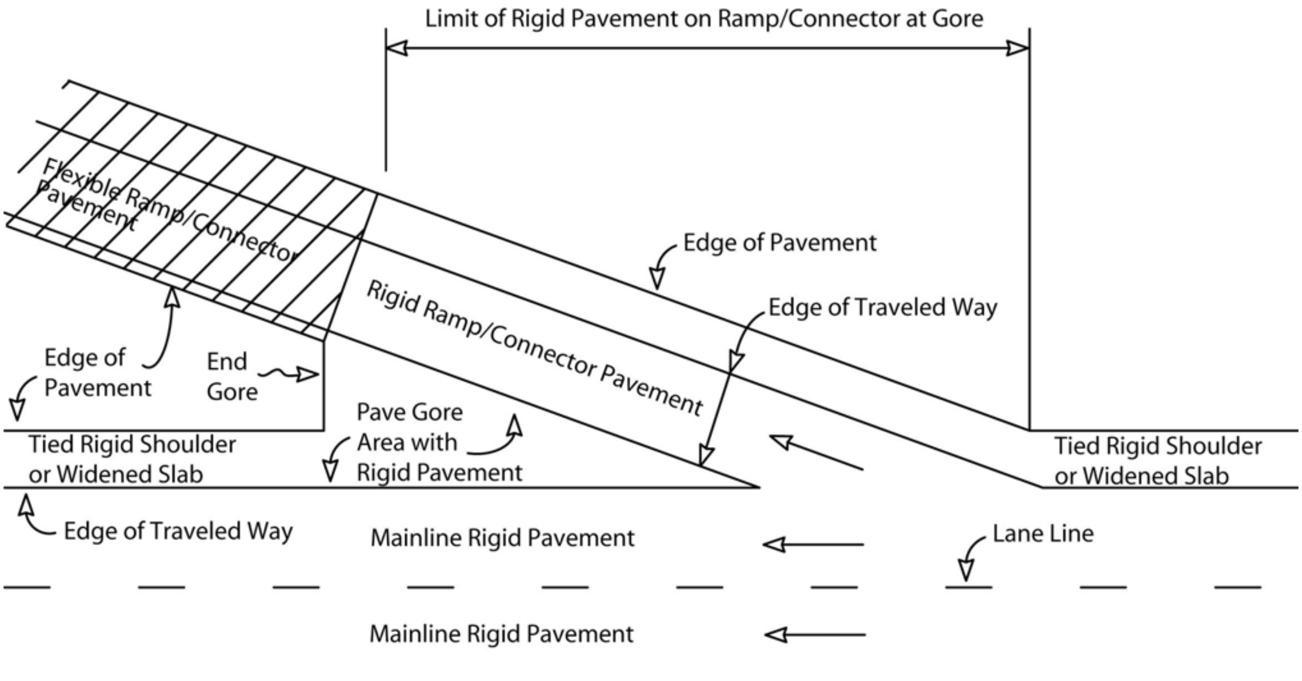
The types of shoulders that are used for rigid pavements are shown in Figure 626.2A and can be categorized into the following three types:

(1) *Tied Rigid Shoulders.* These are shoulders that are built with rigid pavement that are tied to the adjacent lane with tie bars. These shoulders provide lateral support to the adjacent lane, which improves the long-term performance of the adjacent lane, reducing the need for maintenance or repair of the lane. To obtain the maximum benefit, these shoulders should be built monolithically with the adjacent lane (i.e., no contact joints). This will create aggregate interlock between the lane and shoulder, which provides increased lateral support. In order to build the lane and shoulder integrally, the shoulder cross slope needs to match the lane cross slope which may require a design exception (see Index 302.2 for further discussion).

The pavement structure for the tied rigid shoulder should match the pavement structure of the adjacent traffic lane. Special delineation of concrete shoulders may be required to deter the use of the shoulder as a traveled lane. District Traffic Operations should be consulted to determine the potential

Figure 626.1

Rigid Pavement at Ramp or Connector Gore Area



Notes: 1) Not all details shown
2) Off ramp shown. Same conditons apply for on ramps.

need for anything more than the standard edge stripe.

Tied rigid shoulders are the most adaptable to future widening and conversion to a lane. They should be the preferred shoulder type when future widening is planned within the design life of the pavement or where the shoulder will be used temporarily as a truck or bus lane. Where the shoulder is expected to be converted into a traffic lane in the future, the shoulder should be built to the same geometric and pavement standards as the lane. Additionally, the shoulder width should match the width of the future lane.

(2) *Widened slab.* Widened slabs involve constructing the concrete panel for the lane adjacent to the shoulder 4.27 m wide in lieu of the prescribed lane width. The additional width becomes part of the shoulder width and provides lateral support to the adjacent lane. Widened slabs provide as good or better lateral support than tied rigid shoulders at a lower initial cost provided that trucks and buses are kept at least 0.6 m from the edge of the slab. A rumble strip or a raised pavement marking next to the pavement edge line of widened concrete slabs helps discourage trucks and buses from driving on the outside 0.6 meters of the slab. The use of rumble strips or raised markings requires approval from District Traffic Operations.

Widened slabs are most useful in areas where lateral support is desired but future widening is not anticipated or where there is a need to have a different cross slope on the shoulder than that of the adjacent lane.

(3) *Untied Shoulders.* Untied shoulders are flexible shoulders that are not built with a widened slab or rigid shoulders that are not tied to the adjacent lane and not built adjacent to a widened slab. These shoulders do not provide lateral support to the adjacent lane. Although non-supporting shoulders may have lower initial costs, they do not perform as well as tied rigid shoulders or widened slabs, which can lead to higher maintenance costs, user delays, and life cycle costs.

(4) *Selection Criteria.* It is preferred that shoulders be constructed of the same material as the traveled way pavement (in order to facilitate construction, improve pavement performance, and reduce maintenance cost). However, shoulders adjacent to rigid pavement traffic lanes can be either rigid or flexible with the following conditions:

(a) Tied rigid shoulders shall be used for:

- **Rigid pavements constructed in the high mountain and high desert climate regions (see climate map in Topic 615).**
- **Paved buffers between rigid High-Occupancy Vehicle (HOV) lanes and rigid mixed flow lanes. Same for High-Occupancy Toll (HOT) lanes.**
- **Rigid ramps to and from truck inspection stations.**

(b) Either tied rigid shoulders or widened slabs shall be used for:

- **continuously reinforced concrete pavement.**
- **horizontal radii 90 m or less.**
- **Truck and bus only lanes.**

Where tied rigid shoulders or widened slabs are used, they shall continue through ramp and gore areas (see Figure 626.2B).

Because heavy trucks cause deterioration by repeated heavy loading on the outside edge of pavement, at the corners, and the midpoint of the slab, widened slabs or tied rigid shoulders should be used for heavy truck routes with a TI greater than or equal to 14.0.

In those instances where flexible shoulders are used with rigid pavement, the minimum flexible shoulder thickness should be determined in accordance with Topic 633.

These conditions apply to all rigid pavement projects including new construction, reconstruction, widening, adjacent lane replacements, and shoulder replacements. Typically existing flexible shoulders next to rigid pavement are not replaced for rehabilitation projects that involve only grinding, dowel bar retrofits, and individual slab replacements. Consideration should be given to replacing flexible shoulders with tied rigid shoulders or widened slabs when the adjacent lane is being replaced or overlaid with a rigid pavement. The District determines when an existing flexible shoulder is replaced with a rigid shoulder or widened slab.

The shoulder pavement structure selected must meet or exceed the pavement design life standards in Topic 612. In selecting whether to construct rigid or flexible shoulders the following factors should be considered:

- Life-cycle cost of the shoulder.
- Ability and safety of maintenance crews to maintain the shoulder. In confined areas, such as in front of retaining walls or narrow shoulders, and on high volume roadways (AADT > 150,000) consideration should be given to engineering a shoulder that requires the least amount of maintenance, even if it is more expensive to construct.
- Future plans to widen the facility or convert the shoulder to a traffic lane.
- Width of shoulder. When shoulder widths are less than 1.5 meters, tied rigid shoulders are preferable to a widened rigid slab and narrow flexible shoulder, less than 0.9 m, for both constructibility and maintainability.
- For projects where the tracking width lines are shown to encroach onto paved shoulders or any portion of the gutter pan, tied rigid shoulders and the gutter pan structure must be engineered to sustain the weight of the design vehicle. See Topic 404 for design vehicle guidance.

See Index 1003.6(2) for surface quality guidance for highways open to bicyclists.

626.3 Intersections

Standard joint spacing patterns found in the Standard Plans do not apply to intersections. Special paving details for intersections need to be included in the project plans. Special consideration needs to be given to the following features when engineering a rigid pavement intersection:

- Intersection limits
- Joint types and joint spacing
- Joint patterns
- Slab dimensions
- Pavement joints at utilities
- Dowel bar and tie bar placement

Additional information and training is available on the Department Pavement website.

626.4 Roadside Facilities

(1) *Safety Roadside Rest Areas and Vista Points.* If rigid pavement is selected for some site-specific reason(s), the pavement structures used should be sufficient to handle projected loads at most roadside facilities. To select the pavement structure, determine the Traffic Index either from traffic studies and projections developed for the project or the values found in Table 613.5B, whichever is greater. Then select the appropriate pavement structure from the catalog in Index 623.1.

    Joint spacing patterns found in the Standard Plans do not apply to parking areas. Joint patterns should be engineered as square as possible. Relative slab dimensions should be approximately 1:1 to 1:1.25, transverse-to-longitudinal. Transverse and longitudinal joints should be perpendicular to each other. Joints are doweled in one direction and tied in the other in accordance with Index 622.4. Special attention should be given to joint patterns around utility covers and manholes.

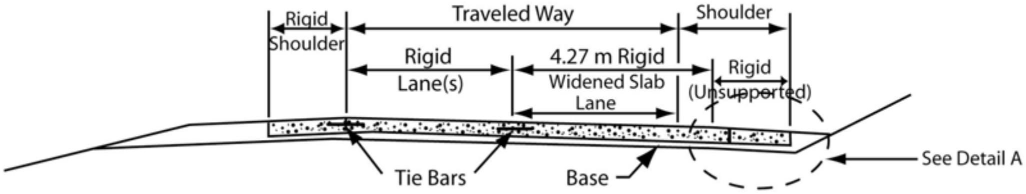
    Use guidelines for intersections in Index 626.3 for further information.

(2) *Park and Ride Facilities.* Flexible pavement should be used for park and ride facilities. If transit buses access the park and ride facility, use the procedures for bus pads in this Index for engineering bus access.

(3) *Bus Pads.* Bus pads are subjected to similar stresses as intersections; however, it is not practical to engineer rigid bus pads according to the Traffic Index, or according to bus

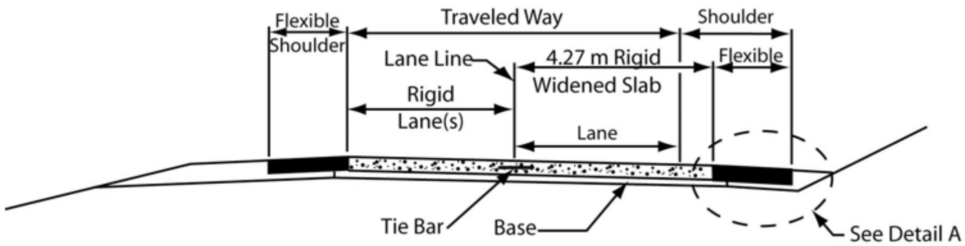
Figure 626.2A

Rigid Pavement and Shoulder Details



RIGID SHOULDERS

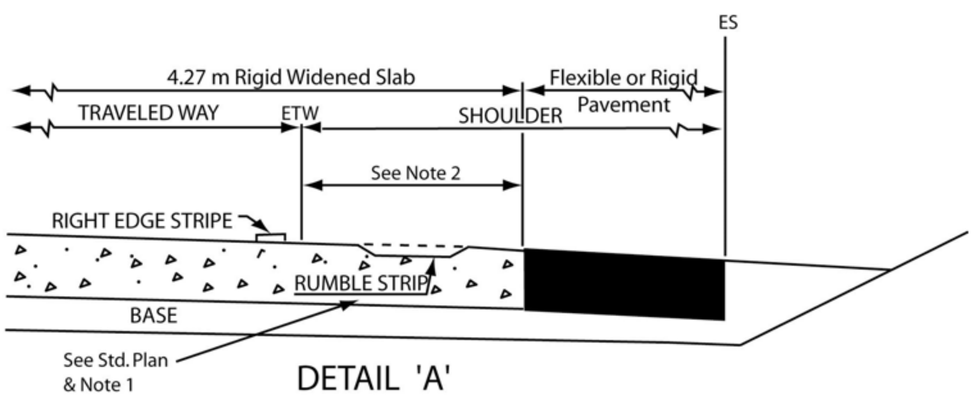
No Scale



FLEXIBLE SHOULDERS

No Scale

NOTE: These illustrations are only to show nomenclature and are not to be used for geometric cross section details.

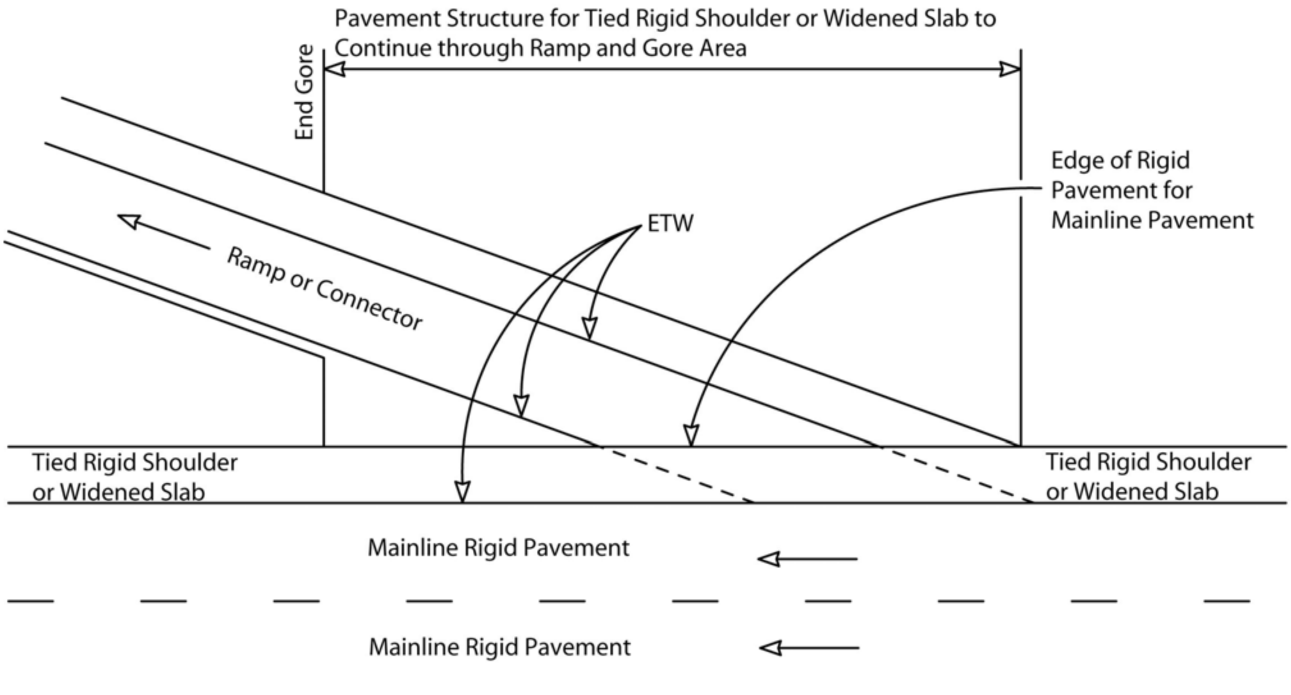


DETAIL 'A'

NOTES:
1. Use of Rumble Strips is determined in consultation with District Traffic Operations.
2. 670 mm for 3.6 meter lane.
   610 mm for 3.66 meter lane.
3. Right side widened slab is shown. Left side widened slab is similar.

Figure 626.2B

Rigid Shoulders Through Ramp and Gore Areas



Notes: 1) Not all details shown
2) Off ramp shown. Same conditions apply for on ramps.

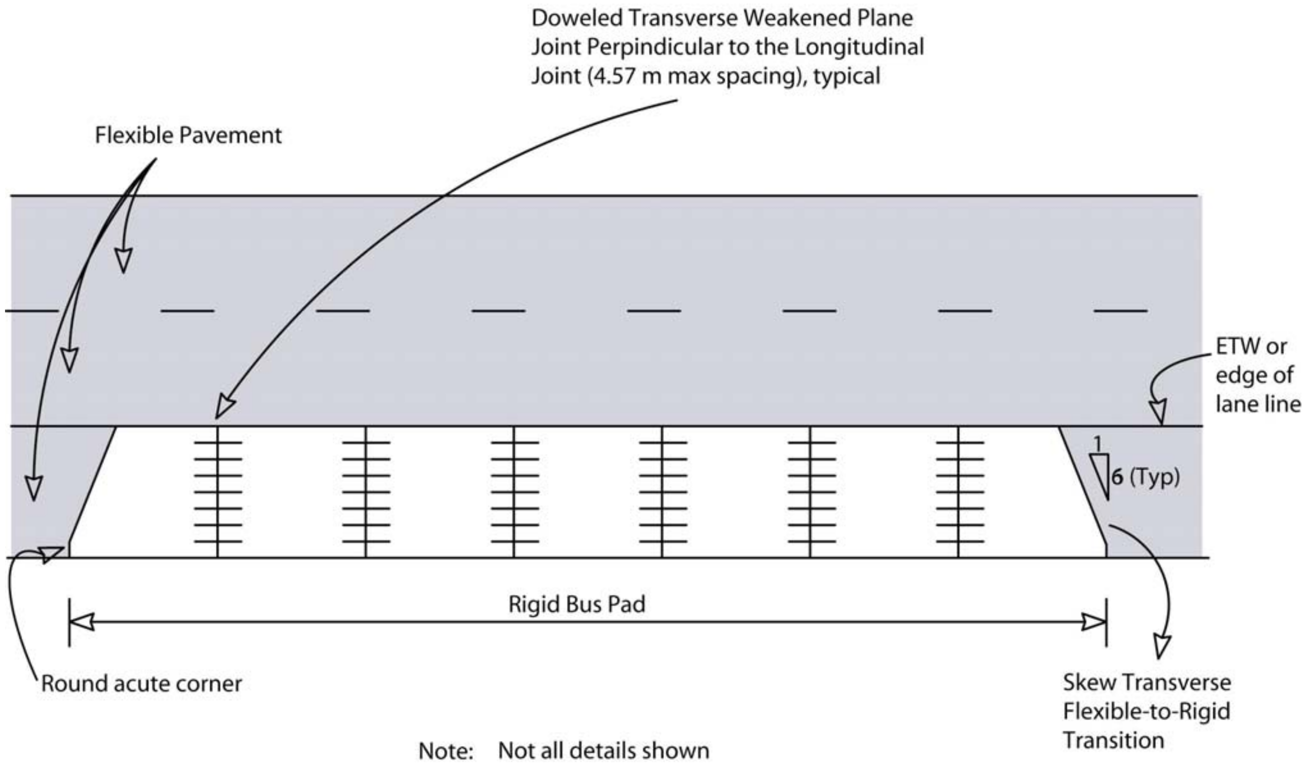
counts. The minimum pavement structure for bus pads should be 255 mm JPCP with dowel bars at transverse joints on top of 150 mm lean concrete base or Type A hot mix asphalt (230 mm CRCP may be substituted for 255 mm JPCP). For Type II soil as described in Table 623.1A, include 150 mm of aggregate subbase. 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 1.2 m. 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 35 m length of bus pad (which is approximately 250% to 300% times the length of typical 12 m 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 4.57 m spacing, but consider skewing (at 6:1 typical) entrance/exit transverse flexible-to-rigid transitions, note that since acute corners can fail prematurely, acute corners should be rounded (see Figure 626.4). Special care should be taken to assure skid resistance in conformance with current Standard Specifications in the braking area, especially where oil drippage is concentrated.

Figure 626.4

Rigid Bus Pad



Note: Not all details shown

CHAPTER 630
FLEXIBLE PAVEMENT

Topic 631 - Types of Flexible Pavements & Materials

Index 631.1 Hot Mix Asphalt (HMA)

HMA consists of a mixture of asphalt binder and a graded aggregate ranging from coarse to very fine particles. The aggregate can be treated and the binder can be modified. HMA could be made from new or recycled material. Examples of recycled asphalt include, but are not limited to, hot and cold in-place recycling. HMA is classified by type depending on the specified aggregate quality and mix design criteria appropriate for the project conditions. HMA types are found in the Standard Specifications and Standard Special Provisions.

631.2 Open Graded Friction Course (OGFC)

OGFC (formerly known as open graded asphalt concrete (OGAC)) is a non-structural wearing course used primarily on HMA. It is occasionally used with modified binders on rigid pavements. The primary benefit of using OGFC is the improvement of wet weather skid resistance, reduced potential for hydroplaning, reduced water splash and spray, and reduced night time wet pavement glare. Secondary benefits include better wet-night visibility of traffic lane stripes and pavement markers, and better wet weather (day and night) delineation between the traveled way and shoulders.

For information and applicability of OGFC in new construction and rehabilitation projects refer to OGFC Guideline available on the Department pavement website. Also, see Maintenance Technical Advisory Guide (MTAG) for additional information and use of OGFC in pavement presentation.

631.3 Rubberized Hot Mix Asphalt (RHMA)

Rubberized asphalt is formulated by mixing granulated (crumb) rubber with hot asphalt to form an elastic binder with less susceptibility to temperature changes. The rubberized asphalt is substituted for the regular asphalt as the binder for the flexible pavement. This is called the wet method. Other methods of using rubber in flexible pavements are available. See Asphalt Rubber Usages Guide (ARUG), available on the Department Pavement website, for further details.

RHMA is generally specified to retard reflection cracking, resist thermal stresses created by wide temperature variations and add flexibility to a structural overlay. At present, the Department uses gap-graded (RHMA-G) and open-graded (RHMA-O) rubberized asphalt. The difference between the two is in the gradation of the aggregate. RHMA-O is used only as a non-structural wearing course. RHMA-G can be used as either a surface course or a non-structural wearing course. RHMA should be considered the strategy of choice when evaluating alternatives for a project. If RHMA is found to be inappropriate due to availability, constructibility, environmental factors, or cost, it shall be documented in the scope document, Project Initiation Document (PID), or Project Report (PR).

The minimum thickness for RHMA (any type) should be 30 mm for new construction and rehabilitation. For pavement preservation, RHMA may be placed as thin as 25 mm provided compaction requirements can be met. The maximum thickness for RHMA-G is 60 mm. The maximum thickness for RHMA-O is 45 mm. If a thicker surface layer or overlay is called for, then a HMA layer should be placed prior to placing the RHMA. RHMA should only be placed over a flexible or rigid surface course and not on a granular layer. RHMA-O may be placed on top of new RHMA-G. Do not place conventional HMA or OGFC over new RHMA pavement.

It is undesirable to place RHMA-G or RHMA-O in areas that will not allow surface water to drain. As an example, a surface that is milled only on the traveled way and not on the shoulder forms a "bathtub" section that can trap water beneath the surface of the traveled way. To prevent this effect, RHMA-G should be placed over the whole cross section of the road (traveled way and shoulders).

For additional information and applicability of RHMA in new construction and rehabilitation projects refer to Asphalt Rubber Usage Guide available on the Department Pavement website.

631.4 Other Types of Flexible Pavement

There are other types of flexible pavements such as cold mix, Resin Pavement, and Sulphur Extended Hot Mix Asphalt. The other types of pavements are either used for maintenance treatments or not currently used on State highways. For pavement preservation and other maintenance treatments refer to the Department's Maintenance Manual.

631.5 Stress Absorbing Membrane Interlayers (SAMI)

SAMI are used with flexible layer rehabilitation as a means to retard reflective cracks, prevent water intrusion, and (in the case of SAMI-R (rubberized)) enhance pavement structural strength. Two types of SAMI are:

- Rubberized (SAMI-R). SAMI-R is a rubberized chip seal.
- Fabric (SAMI-F). SAMI-F, also called Geotextile Pavement Interlayer, consists of asphalt-imbued geotextile.

Judgment is required when considering the use of SAMI.

- Consideration should be given to areas that may prohibit surface water from draining out the sides of the overlay, thus forming a "bathtub" section.
- Since SAMI-R can act as a moisture barrier, they should be used with caution in hot environments where they could prevent underlying moisture from evaporating.
- When placed on an existing pavement, preparation is required to prevent excess stress on the membrane. This includes sealing cracks wider than 5 mm and repairing potholes and localized failures.

A SAMI may be placed between layers of new flexible pavement, such as on a leveling course, or on the surface of an existing flexible pavement. A SAMI-F should not be placed directly on coarse surfaces such as a chip seal, OGFC, areas of numerous rough patches or on a pavement that has been cold planed. Coarse surfaces may penetrate the fabric and/or the paving asphalt binder used to saturate the fabric may be "lost" in the voids or valleys leaving areas of the fabric dry. For the SAMI-F to be effective in these areas, use a layer of HMA prior to the placement of the SAMI-F.

SAMI-F's have been found to be ineffective in the following applications:

- When placed under rubberized hot mix asphalt. This is due to the high placement temperature of the RHMA-G mix, which is close to the melting temperature of the fabric.
- For providing added structural strength when placed in combination with new flexible pavement.
- In the reduction of thermal cracking of the new flexible pavement overlay.

Topic 632-Engineering Criteria

632.1 Engineering Properties

(1) *Smoothness.* The smoothness of a pavement impacts its ride quality, overall durability, and performance. Ride quality (which is measured by the smoothness of ride) is also the highest concern listed in public surveys on pavement condition. Smoothness specifications have been improved and incentive/disincentive specifications have been developed to assure designed smoothness values are achieved in construction. Incentive / disincentive specifications can be used where the project meets the warrants for the specification. For up to date and additional information on smoothness and the application of the smoothness specifications see the smoothness page on the Department pavement website.

(2) *Asphalt Binder Type*. Asphalt binders are most commonly characterized by their physical properties. An asphalt binder's physical properties directly relate to field performance. Although asphalt binder viscosity grading is still common, new binder tests and specifications have been developed to more accurately characterize temperature extremes which pavements in the field are expected to withstand. These tests and specifications are specifically designed to address three specific pavement distress modes: permanent deformation (rutting), fatigue cracking, and low temperature cracking.

In the past, the Department has classified unmodified asphalt binder using viscosity grading based on the Aged Residue (AR) System and Performance Based Asphalt (PBA) binder system. Beginning January 1, 2006, the Department switched to the nationally recognized Performance Graded (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 > 20°C	Placement Temperature ≤ 20°C	
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.

632.2 Performance Factors

The procedures and practices found in this chapter are based on research and field experimentation undertaken by the Department and AASHTO. These procedures were calibrated for pavement design lives of 10-20 years and Traffic Index (TI) ranging from 5.0 to 12. Extrapolations and supplemental requirements were subsequently developed to address longer pavement design lives and higher traffic indices. Details on mix design and other requirements for these procedures are provided in the Standard Specifications and Standard Special Provisions. Alterations to the requirements in these documents can impact the performance of the pavement structure and the performance values found in this chapter.

Topic 633- Engineering Procedures for New & Reconstruction Projects

633.1 Empirical Method

The data needed to engineer a flexible pavement are California R-value of the subgrade and the TI for the pavement design life. Engineering of the flexible pavement is based on a relationship between the gravel equivalent (GE) of the pavement structural materials, the TI, and the California R-value of the underlying material. The relationship was developed by the Department through research and field experimentation.

The procedures and rules governing flexible pavement engineering are as follows, (Sample calculations are provided in the Department Pavement website.):

(1) *Procedures for Engineering Multiple Layered Flexible Pavement.*

(a) The TI is determined to the nearest 0.5 per Index 613.3, and the California R-value is established per Index 614.3.

(b) The gravel equivalent (GE) is defined as the required gravel thickness needed to carry a load compared to a different material's ability to carry the same load.

The following equation is applied to calculate the GE requirement of the entire flexible pavement or each layer is calculated using the following equiation:

$$GE = 0.975(TI)(100 - R)$$

where:

GE = gravel equivalent in mm

TI = Traffic Index

R = California R-value of the material below the layer or layers for which the GE is being calculated.

The GE to be provided by each type of material in the pavement is determined for each layer, starting with the surface layer and proceeding downward. For pavements that include base and/or subbase, a safety factor of 60 mm is added to the GE requirement for the surface layer to compensate for construction tolerances allowed by the contract specifications. Since the safety factor is not intended to increase the GE of the overall pavement, a compensating thickness is subtracted from the subbase layer (or base layer if there is no subbase). For pavements that are full depth asphalt, a safety factor of 30 mm is added to the required GE of the flexible pavement. When determining the appropriate safety factor to be added, Hot Mix Asphalt Base (HMAB) and Asphalt Treated Permeable Base (ATPB) should be considered as part of the surface layer.

(c) The gravel factor (G_f) is the relative strength of a material to gravel. Gravel factors for HMA decrease as TI increases, and also increase with HMA thickness greater than 150 mm; while G_f for base and subbase materials are only dependent on the material type.

The G_f of HMA varies with layer thickness (t) for any given TI as follows:

$t \leq 150$ mm:	$G_f = \frac{5.67}{(TI)^{1/2}}$
$t > 150$ mm:	$G_f = (1.04)\frac{(t)^{1/3}}{(TI)^{1/2}}$

These equations are valid for TIs ranging from 5 to 15. For TIs greater than 15, use a rigid or composite pavement or contact the Office of Pavement Design (OPD) for experimental options. For TIs less than 5, use a TI=5.

(d) The thickness of each material layer is calculated by dividing the GE by the appropriate gravel factor, or from Table 633.1. Typical gravel factors for HMA of thickness equal to or less 150 mm, and various types of base and subbase materials, are provided in Table 633.1. This table also shows the limit thickness for placing HMA for each TI, and the limit thickness for each type of base and subbase materials. Additional information on G_f for base and subbase materials are provided in Table 663.1B.

$$\text{Thickness (t)} = \frac{GE}{G_f}$$

Minimum thickness of any asphalt layer should not be less than twice the maximum aggregate size. When selecting the layer thickness, the value is rounded to the nearest 15 mm. A value midway between 15 mm increments is rounded to the next higher value.

The surface course should have a minimum thickness of 45 mm.

Base and subbase materials, other than ATPB, should each have a minimum thickness of 105 mm. When the calculated thickness of base or subbase material is less than the desired 105 mm minimum thickness, either (a) increase the thickness to the minimum without changing the thickness of the overlying layers or (b) eliminate the layer and increase the thickness of the overlying layers to compensate for the reduction in GE.

Generally, the layer thickness of Lime Treated Subbase (LTS) should be limited, with 200 mm as the minimum and 600 mm as the maximum. A surface layer placed directly on the LTS should have a thickness of at least 75mm.

The thicknesses determined by the procedures provided by this equation are not intended to prohibit other combinations and thickness of materials. Adjustments to the thickness of the various materials may be made to accommodate construction restrictions or practices, and minimize costs, provided the minimum thicknesses, maximum thicknesses, and minimum GE requirements (including safety factors), of the subgrade and each layer in the pavement are satisfied.

(1) *Procedures for Full Depth Hot Mix Asphalt.* Full depth hot mix asphalt applies when the pavement structure is comprised entirely of a flexible surface layer in lieu of base and subbase. The flexible surface layer may be comprised of a single or multiple types of flexible pavements including HMA, RHMA, interlayers, special asphalt binders, or different mix designs. Considerations regarding worker safety, short construction windows, the amount of area to be paved, or temporary repairs may make it desirable in some instances to reduce the total thickness of the pavement by placing full depth hot mix asphalt. Full depth hot mix asphalt also is less affected by moisture or frost, does not let moisture build up in the subgrade, provides no permeable layers that entrap water, and is a more uniform pavement structure. Use the standard equation in Index 633.1(1) with the California R-value of the subgrade to calculate the initial GE for the entire pavement structure. Increase this by adding the safety factor of 30 mm to obtain the required GE for the flexible pavement. Then refer to Table 633.1, select the closest layer thickness

for conventional hot mixed asphalt, and determine the adjusted GE that it provides. The GE of the safety factor is not removed in this design. Adjust the final thickness as needed when using other types of materials than hot mixed asphalt.

A Treated Permeable Base (TPB) layer may be placed below full depth hot mix asphalt on widening projects to perpetuate, or match, an existing treated permeable base layer for continuity of drainage. Reduce the GE of the surface layer by the amount of GE provided by the TPB. In no case should the initial GE of the surface layer over the TPB be less than 40 percent of the GE required over the subbase as calculated by the standard engineering equation. When there is no subbase, use 50 for the California R-value for this calculation. In cases where a working table will be used, the GE of the working table is subtracted from the GE of the surface layer as well. A working table is a minimum thickness of material, asphalt, cement, or granular based, used to place construction equipment and achieve compaction requirements when compaction is difficult or impossible to meet.

(2) *Modifications for Pavement Design Life Greater than 20 Years.* The above procedure is based on an empirical method for a twenty-year pavement service life. For pavement design lives greater than twenty-year, in addition to use a TI for that longer service life, provisions should be made to increase material durability and to protect pavement layers from degradation.

The following enhancements shall be incorporated into all flexible pavements with a pavement design life greater than twenty years:

- Use the procedures for full depth hot mix asphalt to determine the minimum thickness for flexible pavement.
- Place a minimum 150 mm of Class 2 Aggregate base underneath the flexible pavement.
- Use a non-structural wearing course (such as OGFC) above the surface layer (minimum 30 mm). See Index 602.1(5) for further details.
- Use rubberized hot mix asphalt (maximum 60 mm) or a PG-PM binder (minimum 60 mm) for the top of the surface layer.

The following enhancements should be incorporated into all flexible pavements with a pavement design life greater than twenty years when recommended by the District Materials Engineer:

- Use higher asphalt binder content for bottom of the surface layer (rich-bottom concept) and using higher stiffness asphalt binder.
- Utilize subgrade enhancement fabrics at the subgrade for California R-values less than 40.
- Use SAMIs within the surface layer.
- Use a separation fabric above granular layers. Note that the fabric used needs to be able to resist construction loads or construction equipment must be able to keep off of the fabric.

(3) *Alternate Procedures and Materials.* At times, experimental procedures and/or alternative materials are proposed as part of the design or construction. See Topic 606 for further discussion.

633.2 Mechanistic-Empirical Method

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

Table 633.1
Gravel Equivalents (GE) and Thickness of Structural Layers (mm)

Actual Layer Thickness (mm) [5]	HMA [1], [2]											Base and Subbase [3]					
	Traffic Index (TI)											TI is not a factor					
	5.0 & below	5.5 6.0	6.5 7.0	7.5 8.0	8.5 9.0	9.5 10.0	10.5 11.0	11.5 12.0	12.5 13.0	13.5 14.0	14.5 15.0	HMAB LCB	CTPB; CTB (Cl. A)	ATPB	CTB (Cl. B)	AB	AS
	G_f (For HMA thickness equal to or less than 150 mm, Gf decreases with TI)[4]											G_f (constant for any base or subbase material irrespective of TI or thickness)					
	GE for HMA layer (mm)											GE for Base or Subbase layer (mm)					
	2.54	2.32	2.14	2.01	1.89	1.79	1.71	1.64	1.57	1.52	1.46	1.9	1.7	1.4	1.2	1.1	1.0
	GE for HMA layer (mm)											GE for Base or Subbase layer (mm)					
45	114	104	96	90	85	81	77	74	71	68	66	--	--	--	--	--	--
60	152	139	128	121	113	107	103	98	94	91	88	--	--	--	--	--	--
75	191	174	161	151	142	134	128	123	118	114	110	--	--	105	--	--	--
90	229	209	193	181	170	161	154	148	141	137	131	--	--	126	--	--	--
105	267	244	225	211	198	188	180	172	165	160	153	200	180	147	126	116	105
120	305	278	257	241	227	215	205	197	188	182	175	228	204	168	144	132	120
135	343	313	289	271	255	242	231	221	212	205	197	257	230	189	162	149	135
150	381	348	321	302	284	269	257	246	236	228	219	285	255	210	180	165	150
165	421	392	362	338	318	301	287	275	264	254	247	314	281	231	198	182	165
180	473	441	407	380	357	338	322	308	296	285	278	342	306	252	216	198	180
195	526	490	453	422	397	377	359	343	329	317	309	371	332	273	234	215	195
210	--	541	500	466	439	416	396	379	363	350	341	399	357	--	252	231	210
225	--	593	548	511	481	456	434	415	399	384	374	428	383	--	270	248	225
240	--	647	597	557	524	497	473	452	434	418	407	456	408	--	288	264	240
255	--	--	647	604	568	538	513	491	471	453	442	485	434	--	306	281	255
270	--	--	698	652	613	581	553	529	508	489	477	513	459	--	324	297	270
285	--	--	--	701	659	625	595	569	546	526	512	542	485	--	342	314	285
300	--	--	--	750	706	669	637	609	585	563	548	570	510	--	360	330	300
315	--	--	--	801	753	714	680	650	624	601	585	599	536	--	378	347	315
330	--	--	--	--	802	759	723	692	664	639	623	--	--	--	--	--	330
345	--	--	--	--	851	806	767	734	705	679	661	--	--	--	--	--	345
360	--	--	--	--	900	853	812	777	746	718	699	--	--	--	--	--	360
375	--	--	--	--	--	901	858	820	787	758	738	--	--	--	--	--	375
390	--	--	--	--	--	949	904	864	830	799	778	--	--	--	--	--	390
405	--	--	--	--	--	998	950	909	873	840	818	--	--	--	--	--	--
420	--	--	--	--	--	--	997	954	916	882	859	--	--	--	--	--	--
435	--	--	--	--	--	--	1045	1000	960	924	900	--	--	--	--	--	--
450	--	--	--	--	--	--	1094	1046	1004	967	942	--	--	--	--	--	--
465	--	--	--	--	--	--	--	1093	1049	1010	984	--	--	--	--	--	--
480	--	--	--	--	--	--	--	1140	1094	1054	1026	--	--	--	--	--	--
495	--	--	--	--	--	--	--	1188	1140	1098	1069	--	--	--	--	--	--
510	--	--	--	--	--	--	--	--	1187	1143	1113	--	--	--	--	--	--
525	--	--	--	--	--	--	--	--	1233	1188	1156	--	--	--	--	--	--
540	--	--	--	--	--	--	--	--	1280	1233	1201	--	--	--	--	--	--
555	--	--	--	--	--	--	--	--	--	1279	1245	--	--	--	--	--	--
570	--	--	--	--	--	--	--	--	--	1325	1290	--	--	--	--	--	--
585	--	--	--	--	--	--	--	--	--	1372	1336	--	--	--	--	--	--
600	--	--	--	--	--	--	--	--	--	--	1382	--	--	--	--	--	--

Notes:

(1) Open Graded Friction Course (conventional and rubberized) is a non-structural wearing course and provides no structural value.

(2) Top portion of HMA surface layer (maximum 60 mm) may be replaced with equivalent RHMA-G thickness. See Topic 631.3 for additional details.

(3) See Table 663.1B for additional information on Gravel Factors (G_f) and California R-values for base and subbase materials.

(4) These G_f values are for TIs shown and HMA thickness equal to or less than 150 mm only. For HMA thickness greater than 150 mm, appropriate G_f should be determined using the equation in Index 633.1(1)(c).

(5) For HMA layer, select TI range, then go down to the appropriate GE and across to the thickness column. For base or subbase layer, select material type, then go down to the appropriate GE and across to the thickness column.

Topic 634 – Engineering Procedures for Flexible Pavement Preservation

634.1 Preventive Maintenance

For details regarding preventive maintenance strategies for flexible pavement, see the "Maintenance Technical Advisory Guide" on the Department Pavement website. Deflection studies are not required for preventive maintenance projects.

634.2 Capital Preventive Maintenance (CAPM)

The standard design for a flexible pavement CAPM project with an International Roughness Index (IRI) less than 2.65 m/km at PS&E is 45 mm overlay for rubberized asphalt pavements and 60 mm for other asphalt binder pavements. The flexible pavement may be rubberized asphalt, conventional asphalt, or some other approved modified binders. A 60 mm overlay of rubberized asphalt may be appropriate in certain circumstances and may be utilized with the concurrence of both the Headquarters Program Advisor and the Headquarters Office of Pavement Design.

For flexible pavement CAPM projects with an IRI greater than 2.68 m/km, the standard design is to place a 75 mm flexible pavement overlay in two lifts. If the necessary ride improvement cannot be adequately addressed within these CAPM parameters, the project should be developed as a roadway rehabilitation project.

Existing pavement may be cold planed up to the depth of the overlay prior to placing the overlays. Situations where cold planing may be beneficial or even necessary are to maintain profile grade, to maintain vertical clearance, or to taper (transition) to match an existing pavement or bridge surface.

A 20 mm to 30 mm non-structural wearing course (such as an open graded friction course) may be added, but is not to be considered part of the overlay requirements.

Deflection studies are not required for CAPM projects. The roadway rehabilitation requirements for overlays (see Index 635.1(1)) and preparation of existing pavement surface (Index 635.1(8)) apply to CAPM projects. Additional details and information regarding CAPM policies and strategies can be found in Design Information Bulletin 81 "Capital Preventive Maintenance Guidelines."

Topic 635- Engineering Procedures for Flexible Pavement and Roadway Rehabilitation

635.1 Empirical Method

(1) *General.* The methods presented in this topic are based on studies for a ten-year pavement design life with extrapolations for twenty-year pavement design life (For pavement design lives greater than twenty years contact the Headquarters Office of Pavement Design).

Because there are potential variations in materials and environment that could affect the performance of both the existing pavement and the rehabilitation strategy, it is difficult to develop precise and firm practices and procedures that cover all possibilities for the rehabilitation of pavements. Therefore, the pavement engineer should consult with the District Materials Engineer and other pertinent experts who are familiar with engineering, construction, materials, and maintenance of pavements in the geographical area of the project for additional requirements or limitations than those listed in this manual.

Rehabilitation strategies are divided into three categories:

- Overlay
- Mill and Overlay
- Remove and Replace

Rehabilitation designs are governed by one of the following three criteria:

- Structural adequacy

- Reflective crack retardation
- Ride quality

On overlay projects, the entire traveled way and paved shoulder shall be ovrelaid. Not only does this help provide a smoother finished surface, it also benefits bicyclists and pedestrians when they are allowed to use the shoulder.

(2) *Data Collection.* Developing a rehabilitation strategy requires collecting background data as well as field data. The Pavement Condition Report (PCR), as-built plans, and traffic information are some of the sources used to prepare rehabilitation strategy recommendations. A thorough field investigation of the pavement surface condition, combined with a current deflection study and coring, knowledge of the subsurface conditions, thicknesses of existing flexible pavement layers, and a review of drainage conditions are all necessary for developing a set of appropriate rehabilitation strategies.

(3) *Deflection Studies.* Deflection studies along with coring data are used to measure the structural adequacy of the existing pavement. A deflection study is the process of selecting deflection test sections, measuring pavement surface deflection, and calculating statistical deflection values as described in California Test Method 356 for flexible pavement deflection measurements. A copy of the test method can be obtained and/or downloaded from the Department Pavement website.

To provide reliable rehabilitation strategies, deflection studies should be done no more than 18 months prior to the start of construction.

a) Test Sections:

Test sections are portions of a roadway considered to be representative of roadway conditions being studied for rehabilitation. California Test Method 356 provides information on selecting test sections and different testing devices. Test sections should be determined in the field based on safe operation and true representation of pavement sections. Test sections can be determined either by the test operator or by the pavement engineer in the field.

Occasionally, a return to a project site may be required for additional testing after reviewing the initial deflection data in the office.

Individual deflection readings for each test section should be reviewed prior to determining statistical values. This review may locate possible areas that are not representative of the entire test section. An example would be a localized failure with a very high deflection. It may be more cost effective to repair the various failed sections prior to rehabilitation. Thus, the high deflection values in the repaired areas would not be included when calculating statistical values for the representative test sections.

b) Mean and 80th Percentile Deflections:

The mean deflection level for a test section is determined by dividing the sum of individual deflection measurements by the number of the deflections:

$$\bar{x} = \frac{\Sigma D_i}{n}$$

where:

$\bar{x}$ = mean deflection for a test section

D_i = an individual measured surface deflection in the test section

n = number of measurements in the test section

The 80th percentile deflection value represents a deflection level at which approximately 80 percent of all deflections are less than the calculated value and 20 percent are greater than the value. Therefore, a strategy based on 80th percentile deflection will provide thicker rehabilitation than using the mean value.

For simplicity, a normal distribution has been used to find the 80th percentile deflection using the following equation:

$$D_{80} = \bar{x} + 0.84s$$

where:

D_{80} = 80th percentile of the measured surface deflections for a test section,

s = standard deviation of all test points for a test section

$$s = \sqrt{\frac{\sum\left(D_i - \bar{x}\right)^2}{n-1}}$$

D_{80} is typically calculated as part of the deflection study done by the test operator. The pavement engineer should verify that the D_{80} results provided by the operator are accurate.

c) Grouping:

Adjacent test sections may be grouped and analyzed together. There may be one or several groups within the project.

A group is a collection of test sections that have similar engineering parameters. Test sections can be grouped if they have all of the following conditions:

- Average D_{80} that vary less than 0.254 mm.
- Average existing hot mix asphalt thickness that vary less than 30 mm.
- Similar base material.
- Similar TI

Once groups have been identified, D_{80} and existing surface layer thickness of each group can be found by averaging the respective values of test sections within that group.

An alternative to the grouping method outlined above is to analyze each test section individually and then group them based on the results of analysis. This way, all the test sections that have similar rehabilitation strategies would fall into the same group.

(4) *Procedures for Rigid Pavement Overlay on Existing Flexible Pavement (Concrete Overlay).* For concrete overlay (sometimes referred to as whitetopping) strategies, only structural adequacy needs to be addressed. To address structural adequacy, use the tables in Index 623.1 to determine the thickness of the rigid layer. The overlay should be thick enough to be considered a structural layer. Therefore, thin or ultra thin concrete layers (< 205 mm) are not qualified as concrete overlay. To provide a smooth and level grade for the rigid surface layer, place a 30 to 45 mm HMA on top of the existing flexible layer.

(5) *Procedures for Flexible Overlay on Existing Flexible Pavement.*

a) Structural Adequacy. Pavement condition, thickness of surface layer, measured deflections, and the projected TI provide the majority of the information used for determining structural adequacy. Structural adequacy is determined using the following procedures and rules:

1) Determine the Tolerable Deflection at the Surface (TDS). The term "Tolerable Deflection" refers to the level beyond which repeated deflections of that magnitude produce fatigue failure prior to the planned TI. TDS is obtained from Table 635.1A by knowing the existing thickness of the flexible layer and TI. For existing flexible pavement over a treated base, use TI and the TDS values in the row for Treated Base (TB) found in Table 635.1A

The existing base is considered treated if it meets all of the following conditions:

- Its depth is equal to or greater than 105 mm.
- The D_{80} is less than 0.381 mm.

Table 635.1A
Tolerable Deflections at the Surface (TDS) in 0.025 mm

Exist. HMA thick. (mm)	Traffic Index (TI)											
	5	6	7	8	9	10	11	12	13	14	15	16
0	1.676	1.295	1.041	0.864	0.737	0.635	0.559	0.483	0.432	0.381	0.356	0.330
15	1.549	1.194	0.965	0.787	0.686	0.584	0.508	0.457	0.406	0.356	0.330	0.305
30	1.448	1.118	0.889	0.737	0.635	0.533	0.483	0.406	0.381	0.330	0.305	0.279
45	1.346	1.041	0.838	0.686	0.584	0.508	0.432	0.381	0.356	0.305	0.279	0.254
60	1.245	0.965	0.787	0.635	0.533	0.457	0.406	0.356	0.330	0.305	0.254	0.254
75	1.168	0.889	0.711	0.610	0.508	0.432	0.381	0.330	0.305	0.279	0.254	0.229
90	1.092	0.838	0.686	0.559	0.483	0.406	0.356	0.305	0.279	0.254	0.229	0.203
105	1.016	0.787	0.635	0.508	0.432	0.381	0.330	0.305	0.254	0.229	0.203	0.203
120	0.940	0.737	0.584	0.483	0.406	0.356	0.305	0.279	0.254	0.229	0.203	0.178
135	0.889	0.686	0.533	0.457	0.381	0.330	0.279	0.254	0.229	0.203	0.178	0.178
150(1)	0.813	0.635	0.508	0.432	0.356	0.305	0.279	0.229	0.203	0.203	0.178	0.152
TB(2)	0.686	0.533	0.432	0.356	0.305	0.254	0.229	0.203	0.178	0.152	0.152	0.127
	5.5	6.5	7.5	8.5	9.5	10.5	11.5	12.5	13.5	14.5	15.5	16.5
0	1.473	1.143	0.940	0.787	0.686	0.584	0.508	0.457	0.406	0.381	0.330	0.305
15	1.346	1.067	0.864	0.737	0.635	0.533	0.483	0.432	0.381	0.356	0.305	0.279
30	1.270	0.991	0.813	0.686	0.584	0.508	0.457	0.406	0.356	0.330	0.279	0.279
45	1.168	0.914	0.762	0.635	0.533	0.483	0.406	0.356	0.330	0.305	0.279	0.254
60	1.092	0.864	0.711	0.584	0.508	0.432	0.381	0.356	0.305	0.279	0.254	0.229
75	1.016	0.813	0.660	0.559	0.483	0.406	0.356	0.330	0.279	0.254	0.229	0.203
90	0.940	0.737	0.610	0.508	0.432	0.381	0.330	0.305	0.279	0.229	0.229	0.203
105	0.889	0.686	0.559	0.483	0.406	0.356	0.305	0.279	0.254	0.229	0.203	0.178
120	0.813	0.660	0.533	0.457	0.381	0.330	0.279	0.254	0.229	0.203	0.203	0.178
135	0.762	0.610	0.508	0.406	0.356	0.305	0.279	0.229	0.229	0.203	0.178	0.152
150(1)	0.711	0.559	0.457	0.381	0.330	0.279	0.254	0.229	0.203	0.178	0.178	0.152
TB(2)	0.610	0.483	0.381	0.330	0.279	0.254	0.203	0.178	0.178	0.152	0.127	0.127

Notes:

(1) For an HMA thickness greater than 150 mm use the 150 mm depth.

(2) Use the TB (treated base) line to represent treated base materials, regardless of the thickness of HMA cover.

- It is rigid pavement, Lean Concrete Base (LCB), or Class A Cement Treated Base (CTB-A)

2) For each group compare the TDS to the average D_{80}. If the average D_{80} is smaller than the TDS, then the existing pavement is structurally adequate and no overlay is needed to meet this requirement

   If the average D_{80} is greater than the TDS, determine the required percent reduction in deflection at the surface (PRD) to restore structural adequacy as follows:

$$\text{PRD} = \frac{AverageD_{80} - TDS}{AverageD_{80}}(100)$$

   where:

   PRD = Percent Reduction in Deflection required at the surface.

   TDS = Tolerable Deflection at the Surface, in mm

   Average D_{80} = mean of 80th percentile of the deflections for each group

3) Determine the additional GE required using the calculated PRD and Table 635.1B. The additional GE is the amount of aggregate subbase (AS that will provide sufficient strength to reduce the deflections to less than the tolerable level.

4) Determine the required overlay thickness by dividing GE by G_f.

$$\text{Thickness (t)} = \frac{\text{GE}}{\text{G}_\text{f}}$$

   Commonly used G_f for asphaltic materials used for flexible pavement rehabilitation are presented in Table 635.1C.

b) Reflective Cracking. The goal of these procedures is to keep existing pavement cracks from propagating to the surface during the pavement design life. Retarding the propagation of cracks from the existing pavement is required component in engineering overlays. The procedures and rules for engineering for reflective cracking are as follows:

1) Determine the minimum thickness required for a 10-year pavement design life. For flexible pavements over untreated bases, the minimum thickness of a HMA overlay with a ten-year design life should be half the thickness of the existing flexible pavement up to 105 mm.

   For flexible pavements over treated bases (as defined in the previous section on structural adequacy), minimum HMA overlay of 105 mm should be used for a ten-year design life.

   Exception: when the underlying material is a thick rigid layer (200 mm or more) such as an overlaid jointed plain concrete pavement that was not cracked and seated, a minimum thickness of 135 mm should be used.

2) Adjust thickness if the pavement design life is different than 10 years. For a twenty-year design life, experience has determined the thickness should be 125 percent of the ten-year thickness for reflective cracking.

3) Adjust overlay thickness for alternative materials.

   A thickness equivalency of not more than 1:2 is given to the RHMA-G when compared to the HMA for reflective crack retardation. The equivalencies are tabulated in Tables 635.1D.

If a SAMI-R is placed under a non-rubberized hot mix asphalt that is engineered for reflective crack retardation, the equivalence of a SAMI-R depends upon the type of base material under the

Table 635.1B

Gravel Equivalence Needed for Deflection Reduction

Percent Reduction In Deflection (PRD or PRM) (1)	GE (in mm) For HMA Overlay Design	Percent Reduction In Deflection (PRD or PRM) (1)	GE (in mm) For HMA Overlay Design
5	6	46	168
6	6	47	174
7	6	48	180
8	6	49	186
9	9	50	192
10	9	51	201
11	12	52	207
12	15	53	213
13	15	54	219
14	18	55	226
15	21	56	232
16	24	57	241
17	27	58	247
18	27	59	253
19	30	60	259
20	34	61	265
21	37	62	271
22	43	63	277
23	46	64	287
24	49	65	293
25	55	66	299
26	58	67	305
27	61	68	311
28	64	69	317
29	70	70	323
30	73	71	332
31	79	72	338
32	85	73	344
33	88	74	351
34	94	75	357
35	101	76	363
36	107	77	372
37	113	78	378
38	116	79	384
39	122	80	390
40	128	81	396
41	134	82	402
42	140	83	408
43	146	84	418
44	155	85	424
45	162	86	430

Note: (1) PRD – Percent Reduction in Deflection at the surface.
PRM – Percent Reduction in deflection at the Milled depth.

existing pavement. When the base is a treated material, a SAMI-R placed under HMA or OGFC is considered to be equivalent to 30 mm of HMA. When the base is an untreated material SAMI-R is equivalent to 45 mm of HMA.

Table 635.1C

Commonly Used G_f for Asphaltic Materials for Flexible Pavement Rehabilitation

Material	G_f (1)
Hot Mix Asphalt Overlay	1.9
Hot Recycled Asphalt	1.9
Cold in-Place Recycled Asphalt	1.5
HMA Below the Analytical Depth (2)	1.4

Notes:

(1) For G_f of bases and subbases see Table 663.1B.

(2) Analytical depth is defined in 635.1(6)(a).

SAMI-F placed under HMA that is engineered for reflective crack retardation provides the equivalent of 30 mm of HMA. This allows the engineer to decrease the new profile grade and also save on HMA materials.

Wearing courses are not included in the thickness used to address reflective cracking.

Thicker sections may be warranted. Factors to be considered that might necessitate a thicker overlay are:

- Type, sizes, and amounts of surface cracks.
- Extent of localized failures.
- Existing performance material and age.
- Thickness and performance of previous rehabilitation.
- Environmental factors.
- Anticipated future traffic loads (Traffic Index).

Table 635.1D

Reflective Crack Retardation Equivalencies (Thickness in mm)

HMA(1)	RHMA-G	RHMA-G over SAMI-R
45	30	
60	30	
75	45	
90	45	
105	• 45 if crack width < 3 mm • 60 if crack width ≥ 3 mm or underlying material CTB, LCB, or rigid pavement	• N/A for crack width < 3 mm • 30 if crack width ≥ 3 mm and underlying material untreated • 45 if crack width ≥ 3 mm and underlying material CTB, LCB, or rigid pavement
135	45 over 45 HMA	60

Note:

(1) See Index 635.1(5)(b) for minimum and maximum HMA thicknesses recommended by the Department for reflection crack retardation on flexible pavements.

As always, sound engineering judgment will be necessary for final decisions. Final decision for when to use more than the minimum requirements found in this manual rests with the District.

c) Ride Quality. Ride quality is evaluated based on the pavement's smoothness. The Department records smoothness as part of Pavement Condition Survey using the International Roughness Index (IRI). According to FHWA, the IRI value that most motorists consider uncomfortable for flexible pavement is 2.68 m/km (170 in/mile.) When IRI measurements are 2.68 m/km (170 in/mile) or greater, the engineer must address ride quality.

To improve ride quality, place a hot mix asphalt overlay thick enough (75 mm minimum) to be placed in two lifts. RHMA-G may be placed in two 30 mm lifts to meet the ride quality requirement. However, if a 30 mm layer cools prior to compaction, this strategy is inappropriate. A wearing course may be included in the ride quality thickness. SAMI's do not have any effect on ride quality.

Ride quality will ultimately govern the rehabilitation strategy if the requirements for structural adequacy and reflective crack retardation are less than 75 mm.

Note that the Standard Specification requires the contractor to place a 75 mm HMA in one layer. However, projects with pavement rehabilitation recommendations based on improving ride quality must specify in the Special Provisions that the overlay is placed in two lifts.

Examples of design calculations for flexible overlay thickness on existing flexible pavement are available on the Department Pavement website.

(6) *Mill and Overlay Procedures.* Mill and Overlay is the removal of part of the surface layer and placement of an overlay. Since existing pavement thicknesses will have slight variations throughout the project length, leave at least the bottom 45 mm of the existing surface layer intact to ensure the milling machine does not loosen the base material or contaminate the recycled mix during hot or cold in-place recycling. If removal of the surface layer and any portion of the base are required, use the procedures for Remove and Replace in Index 635.1(7).

a) Structural Adequacy. The engineering procedures for determining the structural adequacy for Mill and Overlay, are the same as those for overlays found in Index 635.1(1), with the exception of the following:

- TDS is determined using the thickness of the existing pavement prior to milling.
- Deflections are measured at the surface and adjusted to the milling depth.

The engineer must consider milling down to not more than the "analytical depth". As defined by the Department, the "analytical depth" is the least of:

- The milled depth where the Percent Reduction in deflection required at the Milled depth (PRM) reaches 70 percent,
- The milled depth equals 150 mm,
- The bottom of the existing HMA layer.

The percent reduction in deflection required at the milled depth is based on a research study that determined deflections increase by 12 percent for each additional 30 mm of milled depth up to the analytical depth. Once the analytical depth is reached, the existing HMA material below is considered to be of questionable structural integrity and hence is assigned a G_f of 1.4. Since it is not known at what milled depth the 70 percent PRM level or analytical depth will be reached, an iterative type of calculation is required.

Using the thickness of the existing HMA layer, the TI, and base material, determine the TDS from Table 635.1A. The deflection at the milled depth is found from the equation:

$$DM = D_{80} + \left[(12\%) \left(\frac{MillDepth}{30\ mm} \right) (D_{80}) \right]$$

where

D_{80} = 80th Percentile deflections, in mm.

Mill Depth = the depth of the milling in mm.

DM = the calculated Deflection at the Milled depth in mm

Then:

$$PRM = \left(\frac{DM - TDS}{DM} \right) (100)$$

where

PRM = Percent Reduction in deflection required at the Milled depth.

TDS = Tolerable Deflection at the Surface in mm.

Utilizing the calculated PRM value, go to Table 635.1B to get the total GE required to be placed on top of the milled pavement surface. The total GE required to reduce the measured deflection to the tolerable level is a combination of:

- The GE determined from the overlay calculations.
- The GE required to replace the material removed by the milling process.

If the milling goes below the analytical depth, the analysis changes. The additional GE that is required to replace the existing HMA below the analytical depth is calculated by multiplying the G_f of 1.4 by the milled depth below the analytical depth.

Additional GE = [(1.4)(milled depth below the analytical depth)]

To determine the total GE for the overlay, the additional GE below the analytical depth is added to the required GE above the analytical depth (found from Table 635.1B). As stated in Index 633.1(1)(d), the required minimum thickness of the overlay is determined by dividing the total GE by the G_f of the new overlay material.

$$\text{Thickness (t)} = \frac{GE}{G_f}$$

If milled material is to be replaced by Hot Recycled Asphalt (HRA), the overlay thickness is the same as that of HMA since both materials have a G_f of 1.9 (see Table 635.1C).

Due to its low resistance to abrasion, if the milled material is to be replaced with Cold In-Place Recycled Asphalt (CIPRA), the CIPRA layer must be covered with a wearing surface shortly after the recycling process. To determine the required thickness of the cap layer, first determine the GE of the CIPRA layer:

$$GE_{CIPRA} = (\text{CIPRA thickness})(G_{f\,CIPRA})$$

where:

GE_{CIPRA} = Gravel Equivalence of the CIPRA

$G_{f\ CIPRA}$ = Gravel Factor of CIPRA = 1.5 (see Table 635.1C)

Then, subtract the GE_{CIPRA} from the total GE (GE_{TOTAL}) requirement and divide by the G_f of the cap material:

$$\text{Cap Layer Thickness} = \frac{GE_{TOTAL} - GE_{CIPRA}}{G_f}$$

If the cap layer is OGFC, its thickness should not be considered in pavement structure design. It is recommended to round up to get the CIPRA and cap layer thicknesses.

(b) Reflective Cracking. The minimum thickness for reflective cracking is determined using the same procedures used for reflective cracking for overlays found in Index 635.1(5)(b) except that the thickness is determined based on the remaining surface layer rather than the initial surface layer.

(c) Ride Quality. Milling the existing surface and overlaying with new surface course is considered sufficient to smooth a rough pavement.

(7) *Remove and Replace.* The Remove and Replace operation consists of removing the entire surface layer and part or all of the base and subbase material. The entire removed depth is then replaced with a new flexible or rigid pavement structure. The Remove and Replace strategy is most often used when:

- It is not possible to maintain the existing profile grade using Mill and Overlay.
- Existing base and or subbase material is failing and needs to be replaced.
- It is the most cost effective strategy based on life cycle cost analysis.

Remove and Replace covers a variety of strategies. The discussion found here provides some general rules and minimum requirements for Remove and Replace strategies in general. For more specific information see the technical guidance on the Department Pavement web site.

Because the existing surface layer is removed only structural adequacy needs to be addressed for Remove and Replace.

a) Partial Depth Removal. When only a portion of the existing depth is being removed, consideration needs to be given to the strength of the remaining pavement structure. Because the pavement has been stressed and has been subject to contamination from fines and other materials over time, it does not have the same strength (GE) as new material. Currently, for partial depth removals, the most effective engineering method is to determine the theoretical deflection of the remaining material otherwise known as DM. It should be noted that the greater the depth of removal, the less accurate the determination might be of the calculated deflections.

Also, using deflections for Remove and Replace strategies is also less accurate if a bulldozer or a scraper is used to remove the material under the pavement instead of a milling machine. This method of removing material disturbs the integrity of the in-place material from which the deflections were measured.

Because of these issues, the DME may require reduced GE from what is found in this manual or additional pavement thickness. Final determination of what GE is used rests with the District.

It is recommended that if the removal depth is more than 300 mm, determine the pavement thickness and layers use the method for new or reconstructed pavements discussed in Index 633.1. If the pavement structure is being replaced with rigid pavement, the resulting total pavement structure (including existing pavement left in place) cannot be less than the minimum values found in the rigid pavement catalog in Topic 623.

The analysis used for partial depth Remove and Replace with flexible pavement is similar to the Mill and Overlay analysis. The procedures are as follows:

1) Consider milling down to what is called the analytical depth. This is an iterative type of calculation since it is not known at what milling depth the analytical depth will be reached.

2) Use the thickness of the existing HMA layer, the design TI and base material in Table 635.1A to determine the TDS. Then find the DM knowing D_{80} and the mill depth. Use DM and TDS to find the percent reduction in deflection at the milled depth (PRM).

3) Utilizing this calculated PRM value go to Table 635.1B to obtain the GE required to be placed on top of the milled surface. When the milled depth reaches the analytical depth, the analysis changes. The GE for the

material milled below the analytical depth is added to the GE required at the analytical depth. The GE for each layer is calculated by multiplying G_f by the thickness of the layer milled.

4) Determine the required minimum thickness of HMA needed by dividing the sum of the GE's by the G_f of the new HMA (see equation below.)

$$\text{Thickness (t)} = \frac{GE}{G_f}$$

For the Remove and Replace method, use the G_f for the new HMA commensurate with the TI and HMA thickness found in Table 633.1. The total HMA thickness can be solved for each 15 mm of material milled until the desired profile is reached. Round the replacement thickness to the nearest 15 mm.

5) Adjust thicknesses as needed for alternate materials.

b) Full depth removal. When material is removed all the way to the subgrade, the Remove and Replace strategy should be engineered using the same procedures used for new construction found in Index 633.1.

(8) *Preparation of Existing Pavement.* <u>Existing pavement distresses should be repaired before overlaying the pavement.</u> Cracks wider than 5 mm should be sealed; loose pavement removed/replaced; and potholes and localized failures repaired. Undesirable material such as bleeding seal coats or excessive crack sealant should be removed before paving. Existing thermoplastic traffic stripes and raised pavement markers should be removed. Routing cracks before applying crack sealant has been found to be beneficial. The width of the routing should be 5 mm wider than the crack width. The depth should be equal to the width of the routing plus 5 mm. In order to alleviate the potential bump in the overlay from the crack sealant, leave the crack sealant 5 mm below grade to allow for expansion (i.e., recess fill). The Materials Report should include a reminder of these preparations. Additional discussion of repairing existing pavement can be found on the Department Pavement web site.

(9) *Choosing the Rehabilitation Strategy.* The final strategy should be chosen based on pavement life-cycle cost analysis (LCCA). The strategy should also meet other considerations such as constructibility, maintenance, and the other requirements found in Chapter 610.

635.2 Mechanistic-Empirical Method

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

Topic 636 –Other Considerations

636.1 Traveled Way

(1) *Mainline.* No additional considerations.

(2) *Ramps and Connectors.* Rigid pavement should be considered for freeway-to-freeway connectors and ramps near major commercial or industrial areas (TI > 14.0), truck terminals, and all truck weighing and inspection facilities.

(3) *Ramp Termini.* Distress is compounded on flexible pavement ramp termini by the dissolving action of oil drippings combined with the braking of trucks. Separate pavement strategies should be developed for these ramps that may include thicker pavement structures, special asphalt binders, aggregate sizes, or mix designs. Rigid pavement should be considered for exit ramp termini where there is a potential for shoving or rutting. At a minimum, rigid pavement should be used for exit ramp termini of flexible pavement ramps where a significant volume of trucks is anticipated (TI > 12.0). For the engineering of rigid pavement ramp termini, see Index 626.1(3).

636.2 Shoulders

The TI for shoulders is given in Index 613.5(2). See Index 1003.6(2) for surface quality guidance for highways open to bicyclists.

636.3 Intersections

Where intersections have stop control or traffic signals, special attention is needed to the engineering of flexible pavements to minimize shoving and rutting of the surface caused by trucks braking. Separate pavement strategies should be developed for these intersections that may include thicker pavement structures, special asphalt binders, aggregate sizes, or mix designs. Rigid pavement is another alternative for these locations. For additional information see Index 626.3. For further assistance on this subject, contact your District Materials Engineer, METS, Office of Flexible Pavement Materials, or Division of Design, OPD.

636.4 Roadside Facilities

(1) *Safety Roadside Rest Areas.* Safety factors for the empirical method should be applied to the ramp pavement but not for the other areas.

For truck parking areas, where pavement will be subjected to truck starting/stopping and oil drippings which can soften asphalt binders, separate flexible pavement structures which may include thicker structural sections, alternative asphalt binders, aggregate sizes, or mix designs should be considered. Rigid pavement should also be considered.

(2) *Park & Ride Facilities.* To engineer a park and ride facility based on the standard traffic projections is not practicable because of the unpredictability of traffic. Therefore, standard structures, based on anticipated typical load, have been adopted. However, if project site-specific traffic information is available, it should be used with the standard engineering procedures.

The layer thicknesses shown in Table 636.4 are based on previous practices. These pavement structures are minimal, but are considered adequate since additional flexible surfacing can be added later, if needed, without the exposure to traffic or traffic-handling problems typically encountered on a roadway.

(3) *Bus pads.* Use rigid or composite pavement strategies for bus pads.

Table 636.4
Pavement Structures for Park and Ride Facilities

Subgrade Soil California R-value	Thickness of Layers HMA[1] (mm)	Thickness of Layers AB (mm)
< 40	75	0
	45	105
≥ 40	45	0
≥ 60	Penetration Treatment[2]	

Notes:
(1) Place in one lift.

(2) Penetration Treatment is the application of a liquid asphalt or dust palliative on compacted roadbed material. See Standard Specifications.

Topic 637- Engineering Analysis Software

Software programs for engineering flexible pavements using the procedures in this chapter can be found on the Department pavement website. These programs employ the procedures and requirements for flexible pavement engineering enabling the engineer to compare numerous combinations of materials in seeking the most cost effective pavement structure.

CHAPTER 640
COMPOSITE PAVEMENTS

Topic 641 – Types of Composite Pavement

Index 641.1 Flexible Over Rigid Layer

This configuration consists of a flexible layer on top of a rigid surface layer (typically jointed plain concrete pavement or continuous reinforced concrete pavement) where the flexible layer is used to increase the performance of the rigid layer. (Flexible layers over lean concrete base or cement treated base are considered to be flexible pavements for the purposes of this manual.) The function of the flexible layer is to act as a thermal and moisture blanket to reduce the vertical temperature and moisture gradient within the rigid surface layer and decrease the deformation (curling and warping) of concrete slabs. In addition, the flexible layer acts as a wearing course to reduce wearing effect of wheel loads on the rigid surface layer.

Flexible over rigid composite pavements are found most often on older pavements that have had a flexible pavement overlay such as hot mix asphalt, open graded friction course, or rubberized hot mix asphalt, placed over previously built jointed plain concrete pavement (JPCP) or continuously reinforced concrete pavement (CRCP.) New or reconstructed flexible pavements over JPCP or CRCP typically have not been built in the past on State highways because they have been viewed as combining the disadvantages of rigid pavements (higher initial cost) and flexible pavements (more frequent maintenance).

Thin flexible layers (i.e. sacrificial wearing course) have sometimes been placed over JPCP or CRCP to improve ride quality or friction of the rigid layer. Because ride quality and friction can also be improved by grooving or diamond grinding the existing rigid layer, the engineer should perform a life-cycle cost analysis (LCCA) to determine if diamond grinding/grooving or a flexible sacrificial overlay is more cost effective before deciding which option to select.

In some cases such as matching the existing pavement structure when widening, adding truck lanes to an adjacent flexible pavement, or providing a new wearing surface to an old rigid surface layer that is still structurally sound, composite pavements may be an option.

641.2 Rigid Over Flexible Layer

Because of the minimum 205 mm thickness requirements for rigid surface layers, all pavements with a rigid surface are engineered according to the standards and procedures for rigid pavements in Chapter 620.

Topic 642 – Engineering Criteria

642.1 Engineering Properties

The engineering properties found in Index 622.1 for rigid pavement and Index 632.1 for flexible pavement apply to composite pavements. Care should be taken in selecting materials in the flexible layer to resist reflective crack propagation from the underlying rigid layer and facilitate construction of generally thin flexible layers.

642.2 Performance Factors

Flexible layers placed over rigid surface layers need to be engineered and use materials that will meet the following requirements:

(1) *Reflective Cracking*. Joints or cracks from the underlying rigid surface layer should not reflect through the flexible layer for the service life of the flexible layer.

(2) *Smoothness*. The flexible layer should be engineered to provide an initial IRI of 1.0 m/km and maintain an IRI that is less 2.68 m/km throughout its service life.

(3) *Bonding.* A major factor in the effectiveness and service life of the flexible layer is the condition of the bond between the flexible and rigid layers. For a good bonding condition between flexible and rigid layer, the thickness of the flexible layer does not play an important

role in its service life. Therefore, for practical purposes, if there is no thickness requirement from the structural/constructibility point of view, the minimum thickness of the flexible layer should be based on material factors such as, gradation and aggregate structure, type of binder, etc. To achieve the maximum bond consult the District Materials Engineer or Office of Flexible Pavement Materials and Office of Rigid Pavement Materials and Structural Concrete for options on effective bonding between rigid and flexible layers.

For performance factors of rigid pavement, see Index 622.2.

Topic 643 – Engineering Procedures for New Construction and Reconstruction

643.1 Empirical Method

Before deciding to construct a new composite pavement, a LCCA should be completed to determine whether the composite pavement is more cost effective over the long term than flexible or rigid pavement alternatives.

At present, there is no comprehensive procedure to engineer a structural layer of flexible pavement over a rigid surface layer of JPCP or CRCP. Research is under way to provide guidelines for engineering and construction of composite pavements. When engineering composite pavements using JPCP or CRCP, the rigid layer with base and subbase is engineered as a rigid pavement using the procedures in Index 623.1. No reduction is made to the thickness of the rigid layer on account of the flexible overlay. The flexible pavement is treated as a sacrificial wearing course, and thus has no structural value.

When enough information is not available, the thickness requirement for placing a flexible pavement overlay over an old rigid pavement can be used as a conservative thickness for a new pavement.

643.2 Mechanistic-Empirical Method

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

Topic 644 – Engineering Procedures for Pavement Preservation

644.1 Preventive Maintenance

Preventive Maintenance is used to maintain the surface of the flexible layer or to replace thin flexible layers (i.e., non-structural wearing courses) placed over a rigid surface layer. If work is needed to repair the underlying rigid layer, it should be developed as a CAPM (Index 644.2) or roadway rehabilitation (Topic 645) project. Additional information on preventive maintenance of the flexible layer can be found in the “Maintenance Technical Advisory Guide (MTAG)” available on the Department Pavement website.

644.2 Capital Preventive Maintenance (CAPM)

The procedures and designs for composite pavement CAPM projects are the same as those for flexible pavements (see Index 634.2). In the case of previously constructed crack, seat, and flexible overlay projects, it may be beneficial to mill a portion of the existing flexible layer prior to overlaying. Milling will reduce the thickness of the existing cracked pavement and therefore provide added life to the overlay.

The roadway rehabilitation requirements for overlays (see Index 645.1) and preparation of existing pavement surface (Index 645.1(3)) apply to CAPM projects. Additional details and information regarding CAPM policies and strategies can be found in Design Information Bulletin 81 “Capital Preventive Maintenance Guidelines.”

Topic 645 – Engineering Procedures for Pavement and Roadway Rehabilitation

645.1 Empirical Method

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.

Procedures for engineering rehabilitation projects for composite pavement are as follows:

Because the flexible surface layer is considered to have no structural value, only reflective cracking and ride quality need to be considered.

(1) *Reflective cracking*. If the flexible layer is placed over an existing (old) rigid pavement, the thickness is calculated based on the procedure outlined for rigid pavement rehabilitation, mainly for reflective crack retardation. The thickness depends on the design life of the flexible non-structural wearing course, as well as mix gradation, type and percentage of the binder.

    For additional information on rehabilitation of rigid pavements refer to "Rigid Pavement Preservation and Rehabilitation Guidelines" available on the Department Pavement website.

(2) *Ride Quality*. When the smoothness of the existing roadway is 2.68 m/km or greater as measured by the International Ride Index (IRI), a minimum 75 mm flexible layer (60 mm rubberized hot mix asphalt) should be placed. The overall thickness can be a single material or a combination of open graded, dense/gap graded, or SAMI-R material. Note that in some cases, existing pavement will need to be repaired to assure the roadway smoothness will remain below 2.68 m/km throughout the life of the overlay.

(3) *Preparation of the Existing Pavement.* Existing pavement distresses should be repaired before overlaying the pavement. Cracks wider than 5 mm should be sealed. Undesirable material such as bleeding seal coats or excessive crack sealant should be removed before paving. Existing thermoplastic traffic stripes and raised pavement markers should be removed. Spalls in rigid pavement should be repaired and broken slabs or punchouts replaced. Loose flexible pavement should be removed and replaced, and potholes and localized failures repaired. Ideally, existing non-structural wearing courses should be removed and, if needed, underlying pavement repaired prior to placing a new flexible wearing course. In some cases it may be more practical to overlay over the existing layer. (A LCCA of the two options will help determine which of these options is more cost effective. Note that when doing a LCCA, the need to ultimately remove all flexible layers in the future should be identified and included in the costs for the analysis.)

    Routing cracks before applying crack sealant has been found to be beneficial. The width of the routing should be 5 mm wider than the crack width. The depth should be equal to the width of the routing plus 5 mm. In order to alleviate the potential bump in the overlay from the crack sealant, leave the crack sealant 5 mm below grade to allow for expansion (i.e., recess fill). The Materials Report should include a reminder of these preparations. Additional discussion of repairing existing pavement can be found on the Department Pavement website.

645.2 Mechanistic-Empirical Method

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

CHAPTER 660
BASE AND SUBBASE

Topic 661 - Engineering Considerations

Bases and subbases serve as a support for the surface layer and distribute the wheel load to subgrade material.

In addition to functioning as part of the pavement structure, bases and subbases serve the following functions:

- Slow down the intrusion of fines from the subgrade soil into pavement structural layers.
- Minimize the damage of frost action.
- Prevent the accumulation of free water within or below the pavement structure.
- Provide a working platform for construction equipment.

Topic 662 - Base and Subbase Categories

Index 662.1 Aggregate Base and Subbase

Aggregate bases and subbases consist of a combination of sand, gravel, crushed stone and recycled material. They are classified in accordance with their gradation and the amount of fines. The gradation of the aggregates can affect structural capacity, drainage, and frost susceptibility. The quality of aggregate base and subbase material affects the rate of load distribution and drainage.

662.2 Treated Base and Subbase

(1) *Hot Mix Asphalt Base (HMAB).* Depending on the quality of aggregate, HMAB is classified as dense graded Type A or Type B Hot Mix Asphalt, (HMA). Type A is primarily a crushed aggregate, which provides greater stability than Type B. When used with HMA pavement, the HMAB is to be considered as part of the pavement layer. The HMAB will be assigned the same gravel factor, G_f, as the remainder of the HMA in the pavement structure.

(2) *Other Treated Bases and Subbases.* Treated bases and subbases are materials mixed with asphalt, portland cement, or other stabilizing agents to improve the strength or stiffness of granular material. These materials include lean concrete base (LCB), cement treated base (CTB), asphalt treated base (ATB) and lime treated subbase (LTS). CTB has shown poor performance under rigid pavement in the past. CTB exhibit excessive pumping, faulting, and cracking. This is most likely due to impervious nature of the base, which traps moisture and yet can break down and contribute to the movement of fines beneath the slab.

662.3 Treated Permeable Base and Subbase

Treated permeable bases (TPB) provide a strong, highly permeable drainage layer within the pavement structure. The binder material may be either asphalt (ATPB) or portland cement (CTPB). Either of these TPB layers will generally provide greater drainage capacity than is needed. The standard thickness is based primarily on constructability with an added allowance to compensate for construction tolerances. If material other than ATPB and CTPB with a different permeability, it is necessary to check the permeability and adequacy of the layer thickness. TPB must be used in accordance with a positive sub-drainage system per Index 651.2.

Erosion and stripping (water washing away cement paste, binders, and fines) can be an issue for TPB. Research conducted in the 1990s at the University of California Pavement Research Center (UCPRC) indicates that the use of ATPB is highly susceptible to stripping. Because of this, the Department recommends use of standard aggregate base (AB), with a compaction of the HMA layer of at least 93 percent of theoretical Rice maximum, instead of ATPB for new pavement structures. When ATPB is needed, such as to ensure continuity of existing ATPB/CTPB layer and/or provide drainage through the pavement structure, special provisions should be made to ensure that it is not subjected to conditions that will lead to premature structural failure. The following guidelines should be followed when using ATPB on State highway pavement projects.

(1) *Considerations for using ATPB.* The following two conditions warrant consideration to use ATPB layer in the pavement structure:

(a) When widening or adding lanes adjacent to an existing ATPB layer to ensure continuity of existing ATPB layer.

(b) Where there is need to drain excess water through the pavement, such as when the uphill side of pavement does not allow for drainage. However, when practical, it is better in such cases to use sub-surface drainage to carry water to the other side of the roadway rather than drain excess water through an ATPB layer just below the HMA.

(2) *Added features when using ATPB.* The following features are recommended when using ATPB:

(a) Use edge drains or daylight the edges (see Chapter 650).

(b) If using edge drains, be sure that Maintenance is informed and can budget funds for maintaining edge drains. Developing an estimate of maintenance costs to maintain edge drains and Budget Change Proposals may be required to assure edge drains can be maintained.

(c) Try to use permeable backfill in shoulders on sides of edge drain to avoid bathtub effect if edge drain becomes clogged.

(d) Increase binder content to 3 percent (maybe higher)

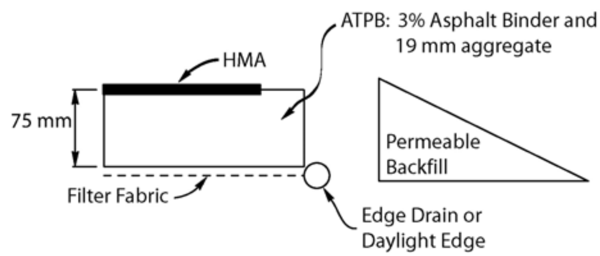
(e) Tack coat each layer.

(f) Perform moisture sensitivity testing on ATPB.

(g) Compaction of the HMA layer should be at least 93 percent of theoretical Rice maximum.

Figure 662.3

Typical Cross Section of ATPB Application



Topic 663 - Engineering Criteria

Because different types of treated and untreated aggregates have different capacities for resisting the forces imposed by traffic loads, this factor must be considered when determining the thickness of the pavement elements. For rigid pavement, this is considered in the design catalogs found in Topic 623. Table 663.1A provides the base and subbase material properties used for the Rigid Pavement Catalog. For flexible pavement, it is accomplished with California R-value and the gravel factor, G_f, which expresses the relative stiffness of various materials when compared to gravel. Table 663.1B provides the California R-values and G_f used for engineering flexible pavements.

The final selection of the bases and subbases for a given project depends on specific factors relative to the available materials, terrain, climate, economics, and past performance of the pavement under similar project or climatic conditions and travel patterns.

Since pavement engineering is a continually evolving field, the District Materials Engineer should be contacted for the latest guidance in base and subbase materials among other related engineering considerations.

Table 663.1A

Base and Subbase Material Properties for Rigid Pavement Catalog

HMA Type A Properties	
Aggregate gradation	0% retained 19 mm sieve 32% retained 9.5 mm 52% retained 4.75 mm sieve 5.5% passing 75 *u*m sieve
Asphalt binder type	See Index 632.1(2) and Table 632.1
Reference temperature	21 °C
Poisson's ratio	0.35
Effective binder content	11.662%
Air voids	8%
Total unit weight	2390 kg/m^3
Thermal conductivity asphalt	1.16 w/m.k
Heat capacity asphalt	0.96 kj/kg.k
Base erodibility index (1)	2
LCB Properties	
Unit weight	2400 kg/m^3
Poisson's ratio	0.20
Elastic Modulus	13,800 MPa
Thermal conductivity	2.16 w/m k
Heat capacity	1.17 kj/kg.k
Base erodibility index (1)	1
AB / AS Properties	
Poisson's ratio	0.40
Coefficient of lateral pressure, K_0	0.5
Resilient Modulus	300/200 MPa
Plasticity Index	1
Passing 75 *u*m	3%
Passing 4.75 mm	20%
D60	8 mm
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/6.9

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 MPa (minimum 2.07 MPa per California Test 373)

(h) Initial cost versus long term maintenance costs for cleanout, repair, traffic control and other pertinent maintenance charges that may be incurred during the life of the facility.

(i) Safety of required maintenance activities, ability to provide maintenance mechanically and to reduce worker exposure.

(j) Inlet and outlet treatment.

(k) Potential for causing erosion and effective water pollution control.

801.6 Use of Drainage References

No attempt has been made herein to detail basic hydrologic and hydraulic engineering techniques.

Various sources of information, including FHWA Hydraulic Engineering Circulars (HEC's); Title 23, Code of Federal Regulations (CFR), Part 650, Subpart A; AASHTO Guidelines; Federal-Aid Policy Guide and numerous hydrology and hydraulics reports and texts have been used to compile this highway drainage guide. Frequent references are made to these publications. Where there is a conflict in information or procedure, engineers must look at all pertinent parameters and use their best judgment, to determine which approach is the most consistent with the objectives of Caltrans drainage design principles and which most closely relates to the specific design problem or project.

Topic 802 - Drainage Design Responsibilities

802.1 Functional Organization

(1) *Division of Design.* The Office of State Highway Drainage Design in Division of Design performs the following functions under the direction of the Headquarters Hydraulics Engineer:

(a) Provide design information, guidance and standards to the Districts for the design of surface and subsurface drainage.

(b) Keep informed on the latest data from research, experimental installations, other public agencies, and industry that might lead to improvement in drainage design practices.

(c) Promote statewide uniformity of design procedures, and the exchange of information between Districts.

(d) Coordinate drainage design practices with other Caltrans Offices.

(e) Review special drainage problems and unusual drainage designs on the basis of statewide experience.

(f) Act in an advisory capacity to the Districts when requested.

(2) *Division of Engineering Services (DES).* The DES is responsible for:

(a) The hydraulic design of bridges, bridge deck drains, and special culverts.

(b) The structural adequacy of all drainage facilities.

(c) The adequacy of pumping plant characteristics and temporary storage. Refer to Topic 839 for further discussion on pumping stations.

(d) Compliance with Federal-Aid Policy Guide, Transmittal 1, G 6012.1 and submittal of preliminary hydraulic data as outlined under Topic 805.

(e) Geotechnical (soil mechanics and foundation engineering) considerations.

(3) *Legal Division.* The Legal Division provides legal advice and guidance to other Caltrans Offices concerning the responsibilities of the Department and owners of property along State highways with regard to surface water drainage.

(4) *Districts.* The District Director is responsible for:

(a) The hydrology for all drainage features except bridges.

(b) The hydraulic adequacy of all drainage features, except bridges and any special culverts and appurtenances designed by the Division of Structures.

(c) Consulting with the Division of Structures when it is proposed that an existing bridge be replaced with a culvert.

(d) Bank and shore protection designs, including erosion protection measures at ends of bridges and other structures designed by the Division of Structures.

(e) Assigning one or more engineers in responsible charge of hydrologic study activities and the hydraulic design of drainage features.

(f) Compliance with current Federal policy for submittal of preliminary plans and hydraulic data for unusual storm drain systems, unusual hydraulic structures, levees and dams formed by highway fills, and major or unusual geotechnical features. See Topic 805.

(g) Providing additional staff as necessary with the training and background required to perform the following:

- Accomplish the objectives of drainage design as outlined under Index 801.4
- Prepare drainage plans or review plans prepared by others.
- Study drainage problems involving cooperative agreements and make recommendations to the decision makers.
- Accumulate and analyze hydrologic and hydraulic data reflecting the local conditions throughout the District for use in design.
- Review drainage changes proposed during construction.
- Make investigations and recommendations on drainage problems arising from the maintenance of existing State highways.
- Coordinate drainage design activities with other District Offices and Branches.
- Coordinate drainage designs with flood control districts and other agencies concerned with drainage by representing the District at meetings and maintaining an active liaison with these agencies at all times.
- Furnish data as required on special problems, bridges, large culverts, culverts under high fills and pumping plants that are to be designed by the Division of Structures.
- Make field inspections of proposed culvert sites, existing drainage structures during storms, and storm damage locations.
- Document condition and file data that might forestall or defend future lawsuits.
- Review permits for drainage facilities to be constructed by other agencies or private parties within the highway right of way.
- Investigate and prepare responses to complaints relative to drainage conditions on or adjacent to the right of way.

Assignment of the duties described above will vary between districts. Due to the increasing complexity of hydraulic and hydrologic issues it is imperative that the more complex analyses be performed by experienced hydraulic designers. To provide guidance on those issues where district hydraulic units should become involved, the following list is provided.

- Storm drain design and calculations.
- Drainage basins exceeding 1.3 km^2.
- Hydrograph development or routing.
- Open channel modification or realignment.
- Retention or detention basins.
- Backwater analysis.
- High potential for flood damage litigation.

- Scour analysis or sediment transport (typically forwarded to DOS).
- Culvert designs greater than 900 mm in diameter.
- Encroachments on FEMA designated floodplains.
- Modifications to inlet or outlet capacities on existing culverts or drainage inlets (e.g., placement of safety end grates, conversion of side opening inlets to grated inlets, etc.).
- Unique hydraulic design features (e.g., energy dissipator design, pumping stations, siphons, etc.).

This list is not all inclusive, and many additional functions are likely to be performed by hydraulic units. Although various constraints may preclude the hydraulic unit from actively performing the design or analysis of these items, a thorough review by that unit should be performed, at a minimum.

(5) *Division of Materials Engineering and Testing Services.* METS provides advice and guidance to other Caltrans Offices and Branches concerning:

(a) Service life, physical properties, and structural adequacy of materials used in drainage design.

(b) Water quality considerations.

802.2 Culvert Committee

The Caltrans Culvert Committee is composed of nine members representing the Offices of State Highway Drainage Design, Structure Design, Office Engineer, and Materials Engineering and Testing Services, along with the Construction Program and Maintenance Program. The Committee is chaired by the Headquarters Hydraulics Engineer in the Office of State Highway Drainage Design. The Committee performs the following functions:

(a) Investigates new materials and new installation methods that may improve the economic service life of culverts and other drainage facilities.

(b) Coordinates drainage design practice with other headquarters departments.

(c) Follows current research and takes steps to implement successful findings.

(d) Acts as an advisory group to Districts and other Caltrans Offices when requested.

(e) Serves as Caltrans liaison with manufacturers, suppliers, contractors and industry associations.

The authority of the Committee is advisory only, and recommendations of the Committee are submitted to the Chief, Division of Design for approval and implementation through design guidelines and standards.

Requests for consideration of new materials, methods, or procedures should be directed to the Committee Chairman.

802.3 Bank and Shore Protection Committee

The Caltrans Bank and Shore Protection Committee is composed of representatives from the ESC Division of Structures Maintenance and Investigation, Office of State Highway Drainage Design, METS, Construction Program, and Maintenance Program. It is chaired by the Office of State Highway Drainage Design representative.

The Committee performs the following functions:

(a) Acts as a service and an advisory group available to Districts and Caltrans Offices and Branches upon written request for special investigations or study. Requests for special investigation of rock slope protection, channel or bridge protection, major channel changes, etc. should be directed to the Committee Chair.

(b) Provides conceptual input and acts as approval authority for supplements or modifications to bank and shore protection practice publications as warranted.

(c) Investigates and provides input toward the development of detailed design criteria for the various types of bank and shore protection.

(d) Observes performances of existing and/or experimental installations during or following severe exposures. The Districts or Caltrans Offices or Branches are requested to inform the Chair, Bank and Shore Protection Committee, or any available members of the Committee, of damage to installations by flood or high seas.

(e) Upon submission by the Department's New Products Coordinator, the Committee evaluates new products and processes related to bank and shore protection for possible approval.

Topic 803 - Drainage Design Policies

803.1 Basic Policy

In drainage design, the basic consideration is to protect the highway against damage from storm and subsurface waters, taking into account the effect of the proposed improvement on traffic and property. Unless the State would benefit thereby, or the cost is borne by others, no improvement in the drainage of areas outside the right of way is to be considered on Caltrans projects.

803.2 Cooperative Agreements

The extent of the department's financial participation in cooperative drainage improvement projects must be commensurate with the benefits to the Department and the traveling public.

(1) Local Agencies. Caltrans may participate with Local Agencies, Flood Control Districts or Drainage Assessment Districts on drainage improvement projects. Such projects must be covered by a formal agreement prepared and processed in accordance with instructions in the Caltrans Cooperative Agreement Manual.

(2) Federal and State Flood Control Projects. The cost of upgrading or modifying existing State highway facilities to accommodate Federal and/or State funded flood control projects is normally the responsibility of the agency funding the project. As necessary, Caltrans may enter into agreements containing provisions that the cost of betterments to existing highways, including drainage features, will be paid for by the Department. The Cooperative Agreement Manual contains procedures for preparing interagency agreements.

803.3 Up-Grading Existing Drainage Facilities

(1) Rehabilitation and Reconstruction Projects. The hydraulic adequacy, as well as the structural adequacy of existing drainage facilities should be evaluated early in the project development process on pavement rehabilitation and highway reconstruction projects.

Repair or replacement of structurally deficient drainage structures and up-grading of hydraulically inadequate drainage facilities should, whenever practicable, be included in the work of the proposed project. A thorough investigation of upstream and downstream conditions is often required to reveal what adverse effects there may be with increasing the capacity or velocity of existing cross drainage.

A cooperative agreement should be negotiated when the proposed work includes the upgrading of an existing storm drain system under the jurisdiction of a local or other public agency.

(2) Proposed Upstream Development. Unless developers of land in the drainage basin upstream of existing State highways incorporate positive stormwater management practices, such as detention or retention storage basins within their improvement areas, the peak flow from stormwater runoff is nearly always increased. As a practical matter, minor increases in peak flow are usually not objectionable. However, uncontrolled upstream development or diversions can significantly increase the peak flow run-off causing the passable capacity of the downstream drainage systems, including existing highway culverts, to be exceeded.

When reasonable solutions to potential drainage problems associated with such increased flows include the up-grading of drainage facilities within the State highway right-of-way, cooperative agreements with the responsible local agency should be negotiated. The local agency having permit authority has the responsibility for assessing liabilities and seeking commensurate funding for mitigation of run-off impacts from the developers. The local agency should not allow potentially harmful developments to proceed until all issues have been resolved. If it becomes apparent that the District, the local agency and the developer may not amiably reach agreement, the matter should be referred to Caltrans Legal Division before there is an impasse in the negotiations.

Caltrans financial participation in such drainage improvements must be based on the general rule stated in Index 803.2 Cooperative Agreements.

(3) *Hydraulically Inadequate Facilities.* Land use changes nearly always cause areas to become less pervious and drainage basins to yield greater volumes and increase peak stormwater run-off flows. Even development of a small parcel of land within a drainage basin causes some increase in stormwater run-off. Individually the increase may be negligible. Collectively these incrementally small increases over time may cause the design capacity of an existing culvert to be exceeded.

The up-grading of this category of hydraulically inadequate drainage facilities may be partially or fully financed by Caltrans. Only if the benefit cost (b/c) ratio is equal to or greater than one is up-grading viable for normal Caltrans project funding. When the benefits to the Department and the traveling public do not justify increasing the capacity, up-grading may still be accomplished cooperatively with the local agency in accordance with the general rule for participation under Index 803.2 Cooperative Agreements.

Topic 804 - Floodplain Encroachments

804.1 Purpose

The purpose of these instructions is to provide uniform procedures and guidelines for Caltrans multi-disciplinary evaluation of proposed highway encroachments on floodplains.

804.2 Authority

Title 23, CFR, Part 650, Subpart A, prescribes FHWA's "...policies and procedures for the location and hydraulic design of highway encroachments on floodplains, ...". The CFR's may be found on-line at: http://www.access.gpo.gov/nara/cfr/cfr-table-search.html

804.3 Applicability

The guidance provided herein establishes Caltrans procedures whenever a floodplain encroachment is anticipated. Adherence to these procedures will also ensure compliance with applicable Federal regulations which apply to any Federally approved highway construction, reconstruction, rehabilitation, repair, or improvement project which affects the (100-year) base floodplain. Work outside the limits of the base floodplain should be reviewed to see if it affects the (100-year) base floodplain. The only exception is repairs made during or immediately following a disaster. The premise is that all Federal-aid projects be evaluated and that diligent efforts be made to:

- Avoid significant floodplain encroachments where practicable.
- Minimize the impact of highway actions that adversely affect the base floodplain.
- Be compatible with the National Flood Insurance Program (NFIP) of the Federal Emergency Management Agency (FEMA).

The following definitions of terms are made for the purpose of uniform application in the documentation and preparation of floodplain evaluation reports. Refer to Title 23, CFR, Part 650, Section 650.105 for a complete list of definitions.

(1) *Base Flood.* The flood or tide having a 1 percent chance of being exceeded in any given year (100-year flood).

(2) *Base Floodplain.* The area subject to flooding by the base flood. Every watercourse (river, creek, swale, etc.) is subject to flooding and theoretically has a base floodplain.

(3) *Design Flood*. The peak discharge, volume if appropriate, stage or wave crest elevation of the flood associated with the probability of exceedance selected for the design of a highway encroachment. By definition, the highway will not be inundated from the stage of the design flood.

(4) *Encroachment.* An action within the limits of the base floodplain. Any construction activity (access road, building, fill slopes, bank or slope protection, etc.) within a base floodplain constitutes an encroachment.

(5) *Location Hydraulic Study.* A term from 23 CFR, Section 650.111 referring to the preliminary investigative study to be made of base floodplain encroachments by a proposed highway action. The extent of investigation and the discussion content in the required documentation of the "Location Hydraulic Study" is very site specific and need be no more than that which is commensurate with the risk(s) and impact(s) particular to the location under consideration. The information developed, documented (refer to Figure 804.7A) and retained in the project file is the suggested minimum necessary for compliance.

(6) *Natural and Beneficial Floodplain Values.* This shall include but is not limited to fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.

(7) *Overtopping Flood.* The flood described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief.

(8) *Regulatory Floodway.* The floodplain area that is reserved in an open manner by Federal, State or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount (not to exceed 0.3 m as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program).

804.5 Procedures

Floodplain evaluations are essentially an extension of the environmental assessment process and instructions contained in the Environmental Handbook and the Project Development Procedures Manual are to be followed. Early in the planning of a project it is necessary to first determine:

(a) If a proposed route alternative will encroach on a base floodplain (refer to Index 804.4 (2)) or,

(b) Where proposed construction on existing highway alignment encroaches on a base floodplain.

A Location Hydraulic Study is used to determine (a) and (b) above. Refer to Index 804.4 (4) and 804.7 (2)(b) for further discussion.

Where National Flood Insurance Program (NFIP) Maps and study reports are available, their use is mandatory in determining whether a highway location alternative will include an encroachment on the base floodplain. Three types of NFIP maps are published which, if available, may be obtained from the District Hydraulics Branch: Flood Hazard Boundary Map (FHBM), Flood Boundary and Floodway Map (FBFM), and Flood Insurance Rate Map (FIRM).

n = Manning's roughness coefficient for sheet flow (see Table 816.6A).

i = Design storm rainfall intensity in mm/h.

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

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

$$T_t = \frac{5.476\ L^{4/5}\ n^{4/5}}{P_2^{1/2}\ \ s^{2/5}}$$

where P_2 is the 2-year, 24-hour rainfall depth in mm (ref. NOAA Atlas 2, Volume XI or use either of the following web site addresses; http://www.wrcc.dri.edu/pcpnfreq.html or, http://www.nws.noaa.gov/oh/hdsc/noaaatlas2.htm).

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

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

Table 816.6A
Roughness Coefficients For Sheet Flow

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

(1) Woods cover is considered up to a height of 30 mm, which is the maximum depth obstructing sheet flow.

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

Average velocities for the Upland Method can be taken directly from Figure 816.6 or may be calculated from the following equation:

$$V = kS^{1/2}$$

Where S is the slope in percent and k (m/s) is an intercept coefficient depending on land cover as shown in Table 816.6B.

Table 816.6B
Intercept Coefficients for Shallow Concentrated Flow

Land cover/Flow regime	**K (m/s)**
Forest with heavy ground litter; hay meadow	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland	0.152
Short grass pasture	0.213
Cultivated straight row	0.274
Nearly bare and untilled-alluvial fans	0.305
Grassed waterway	0.457

The travel time can be calculated from:

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

where T_t is the travel time in minutes, L the length in m, and V the flow velocity in m/s.

(3) *Channel flow travel time.* When the channel characteristics and geometry are known the preferred method of estimating channel flow time is to divide the channel length by the channel velocity obtained by using the Manning equation, assuming bankfull conditions. See Index 864.3, Open Channel Flow Equations for further discussion of Manning's equation.

Appropriate values for "n", the coefficient of roughness in the Manning equation, may be found in most hydrology or hydraulics text and reference books. Table 864.3A gives some "n" values for lined and unlined channels, gutters, and medians. Procedures for selecting an appropriate hydraulic roughness coefficient may be found in the FHWA report, "Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains". Generally, the channel roughness factor will be much lower than the values for overland flow with similar surface appearance.

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

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

Table 819.2B

Runoff Coefficients for Developed Areas

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

The Regional Flood-Frequency equations are applicable only to sites within the flood-frequency regions for which they were derived and on streams with virtually natural flows. For example, the equations are not generally applicable to small basins on the floor of the Sacramento and San Joaquin Valleys as the annual peak data which are the basis for the regression analysis were obtained principally in the adjacent mountain and foothill areas. Likewise, the equations are not directly applicable to streams in urban areas affected substantially by urban development. In urban areas the equations may be used to estimate peak discharge values under natural conditions and then by use of the techniques described in the publication or HDS No. 2, adjust the discharge values to compensate for urbanization. Further limitations on the use of USGS Regional Flood-Frequency equations are:

Region	Drainage Area (A) mi^2	Mean Annual Precip (P) in.	Altitude Index (H) 1000 ft.
(1)North Coast	0.2-3000	19-104	0.2-5.7
Northeast	0.2-25	all	all
Sierra	0.2-9000	7-85	0.1-9.7
Central Coast	0.2-4000	8-52	0.1-2.4
South Coast	0.2-600	7-40	all
(2)South Lahontan-Colorado Desert	0.2-90	all	all

Notes: Values shown in table have not been converted to metric system.

(1) In the North Coast region, use a minimum value of 1 for altitude index (H)
(2) Use upper limit of 25 square miles

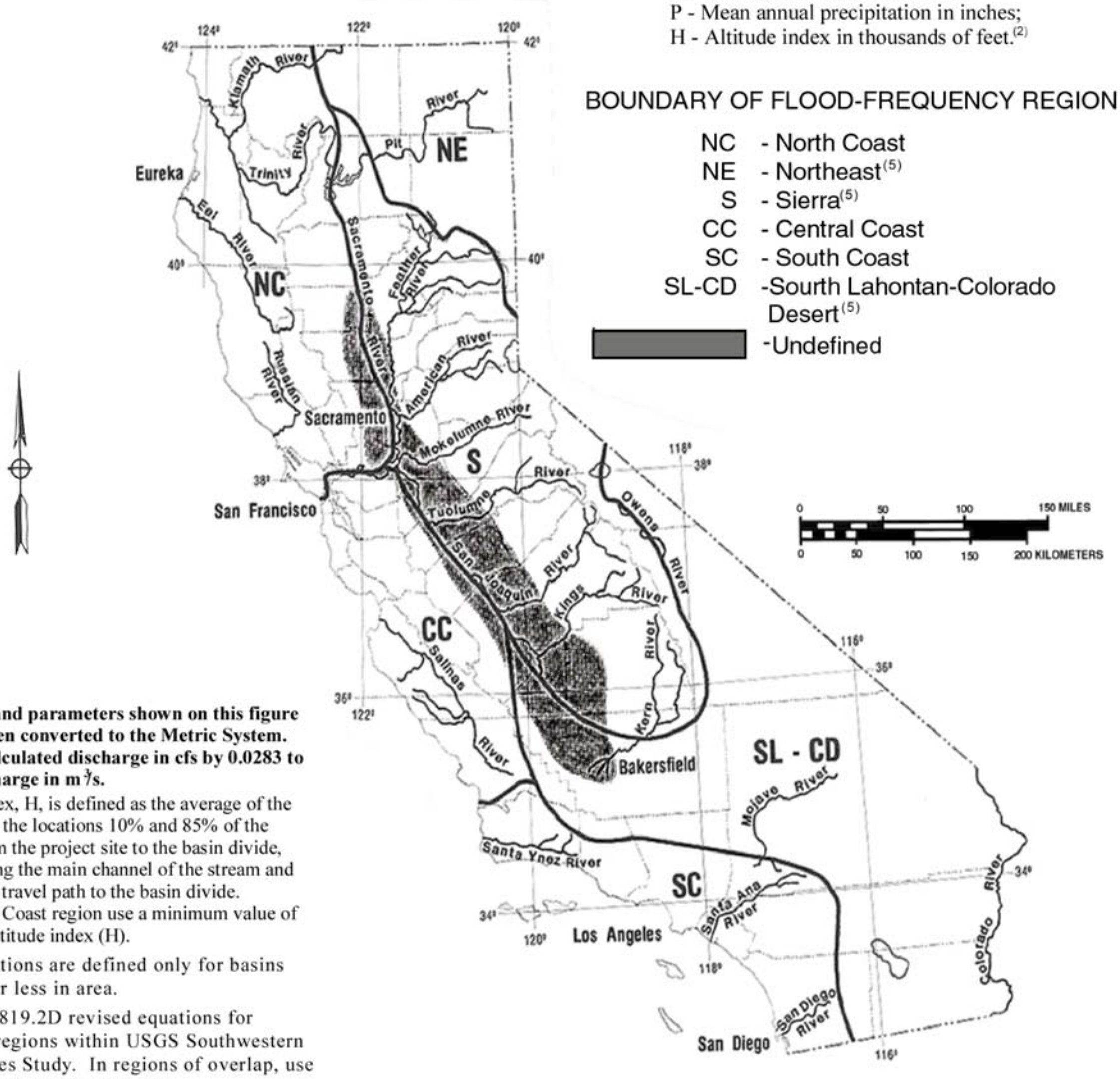
A method for directly estimating design discharges for some gaged and ungaged streams is also provided in HDS No. 2. The method is applicable to streams on or nearby those for which study data are available.

Figure 819.2C
Regional Flood-Frequency Equations (1)

NORTH COAST REGION(3)					NORTHEAST REGION(4)			SOUTH LAHONTAN-COLORADO DESERT REGION(4)		
Q_2	$=3.52$	$A^{0.90}$	$p^{0.89}$	$H^{-0.47}$	Q_2	$=22$	$A^{0.40}$	Q_2	$=7.3$	$A^{0.30}$
Q_5	$=5.04$	$A^{0.89}$	$p^{0.91}$	$H^{-0.35}$	Q_5	$=46$	$A^{0.45}$	Q_5	$=53.0$	$A^{0.44}$
Q_{10}	$=6.21$	$A^{0.88}$	$p^{0.93}$	$H^{-0.27}$	Q_{10}	$=61$	$A^{0.49}$	Q_{10}	$=150$	$A^{0.53}$
Q_{25}	$=7.64$	$A^{0.87}$	$p^{0.94}$	$H^{-0.17}$	Q_{25}	$=84$	$A^{0.54}$	Q_{25}	$=410.0$	$A^{0.63}$
Q_{50}	$=8.57$	$A^{0.87}$	$p^{0.96}$	$H^{-0.08}$	Q_{50}	$=103$	$A^{0.57}$	Q_{50}	$=700.0$	$A^{0.68}$
Q_{100}	$=9.23$	$A^{0.87}$	$p^{0.97}$		Q_{100}	$=125$	$A^{0.59}$	Q_{100}	$=1080.0$	$A^{0.71}$

SIERRA REGION					CENTRAL COAST REGION					SOUTH COAST REGION			
Q_2	$=0.24$	$A^{0.88}$	$p^{1.58}$	$H^{-0.80}$	Q_2	$=0.0061$	$A^{0.92}$	$p^{2.54}$	$H^{-1.10}$	Q_2	$=0.14$	$A^{0.72}$	$p^{1.62}$
Q_5	$=1.20$	$A^{0.82}$	$p^{1.37}$	$H^{-0.64}$	Q_5	$=0.118$	$A^{0.91}$	$p^{1.95}$	$H^{-0.79}$	Q_5	$=0.40$	$A^{0.77}$	$p^{1.69}$
Q_{10}	$=2.63$	$A^{0.80}$	$p^{1.25}$	$H^{-0.58}$	Q_{10}	$=0.583$	$A^{0.90}$	$p^{1.61}$	$H^{-0.64}$	Q_{10}	$=0.63$	$A^{0.79}$	$p^{1.75}$
Q_{25}	$=6.55$	$A^{0.79}$	$p^{1.12}$	$H^{-0.52}$	Q_{25}	$=2.91$	$A^{0.89}$	$p^{1.26}$	$H^{-0.50}$	Q_{25}	$=1.10$	$A^{0.81}$	$p^{1.81}$
Q_{50}	$=10.4$	$A^{0.78}$	$p^{1.06}$	$H^{-0.48}$	Q_{50}	$=8.20$	$A^{0.89}$	$p^{1.03}$	$H^{-0.41}$	Q_{50}	$=1.50$	$A^{0.82}$	$p^{1.85}$
Q_{100}	$=15.7$	$A^{0.77}$	$p^{1.02}$	$H^{-0.43}$	Q_{100}	$=19.7$	$A^{0.88}$	$p^{0.84}$	$H^{-0.33}$	Q_{100}	$=1.95$	$A^{0.83}$	$p^{1.87}$

Q - Peak discharge in CFS, subscript indicates recurrence interval, in years;
A - Drainage area in square miles;
P - Mean annual precipitation in inches;
H - Altitude index in thousands of feet.(2)



NOTES:

(1) Equations and parameters shown on this figure have not been converted to the Metric System. Multiply calculated discharge in cfs by 0.0283 to obtain discharge in m³/s.

(2) Altitude index, H, is defined as the average of the elevations at the locations 10% and 85% of the distance from the project site to the basin divide, measure along the main channel of the stream and the overland travel path to the basin divide.

(3) In the North Coast region use a minimum value of 1.0 for the altitude index (H).

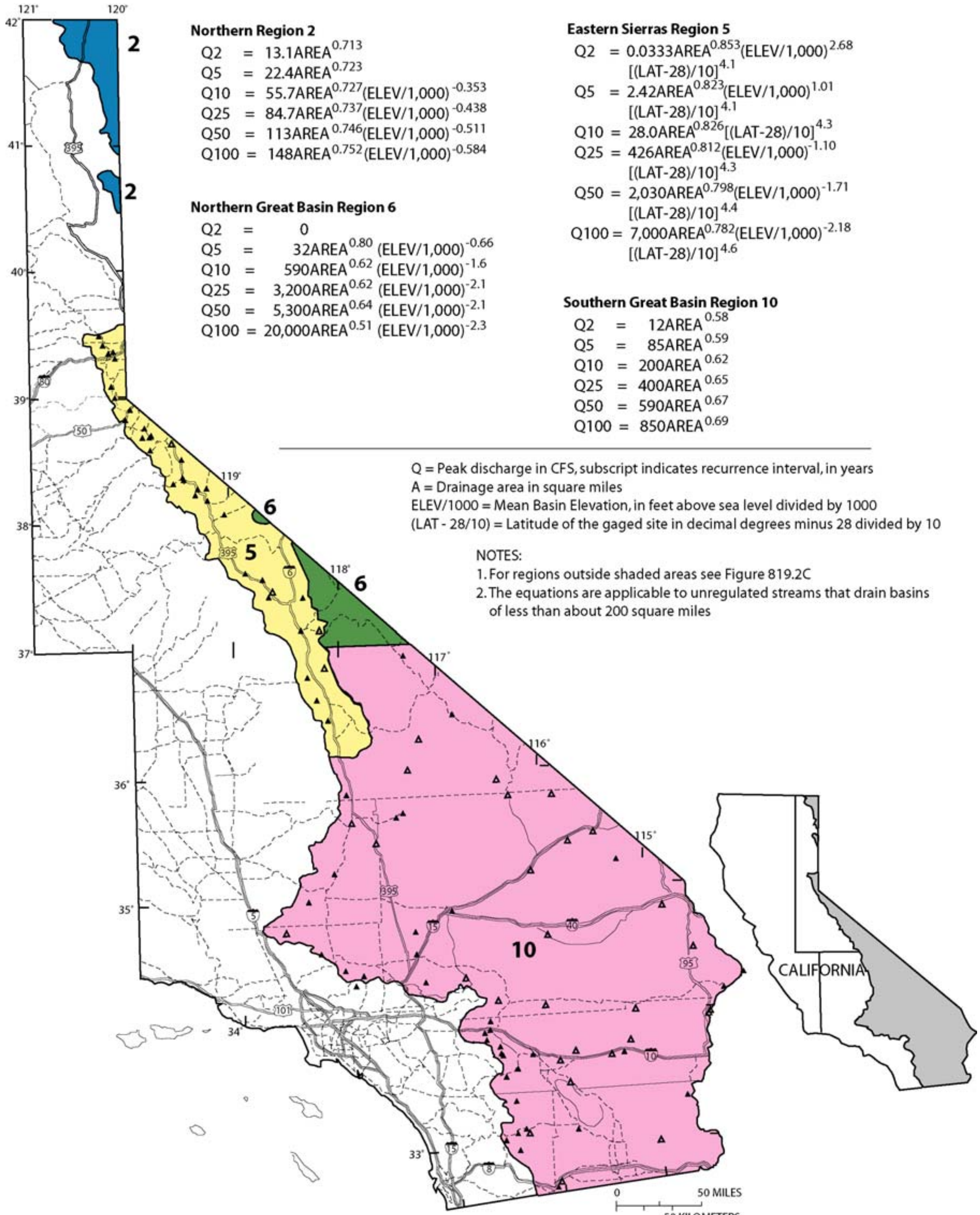
(4) These Equations are defined only for basins of 65 km² or less in area.

(5) See Figure 819.2D revised equations for California regions within USGS Southwestern United States Study. In regions of overlap, use equations from Figure 819.2D.

Figure 819.2D

Regional Flood Frequency Equations for California Regions within USGS Southwestern United States Study*

*USGS Open File Report 93-419 (1994)



Northern Region 2

$Q2 = 13.1AREA^{0.713}$
$Q5 = 22.4AREA^{0.723}$
$Q10 = 55.7AREA^{0.727}(ELEV/1{,}000)^{-0.353}$
$Q25 = 84.7AREA^{0.737}(ELEV/1{,}000)^{-0.438}$
$Q50 = 113AREA^{0.746}(ELEV/1{,}000)^{-0.511}$
$Q100 = 148AREA^{0.752}(ELEV/1{,}000)^{-0.584}$

Northern Great Basin Region 6

$Q2 = 0$
$Q5 = 32AREA^{0.80}(ELEV/1{,}000)^{-0.66}$
$Q10 = 590AREA^{0.62}(ELEV/1{,}000)^{-1.6}$
$Q25 = 3{,}200AREA^{0.62}(ELEV/1{,}000)^{-2.1}$
$Q50 = 5{,}300AREA^{0.64}(ELEV/1{,}000)^{-2.1}$
$Q100 = 20{,}000AREA^{0.51}(ELEV/1{,}000)^{-2.3}$

Eastern Sierras Region 5

$Q2 = 0.0333AREA^{0.853}(ELEV/1{,}000)^{2.68}[(LAT-28)/10]^{4.1}$
$Q5 = 2.42AREA^{0.823}(ELEV/1{,}000)^{1.01}[(LAT-28)/10]^{4.1}$
$Q10 = 28.0AREA^{0.826}[(LAT-28)/10]^{4.3}$
$Q25 = 426AREA^{0.812}(ELEV/1{,}000)^{-1.10}[(LAT-28)/10]^{4.3}$
$Q50 = 2{,}030AREA^{0.798}(ELEV/1{,}000)^{-1.71}[(LAT-28)/10]^{4.4}$
$Q100 = 7{,}000AREA^{0.782}(ELEV/1{,}000)^{-2.18}[(LAT-28)/10]^{4.6}$

Southern Great Basin Region 10

$Q2 = 12AREA^{0.58}$
$Q5 = 85AREA^{0.59}$
$Q10 = 200AREA^{0.62}$
$Q25 = 400AREA^{0.65}$
$Q50 = 590AREA^{0.67}$
$Q100 = 850AREA^{0.69}$

Q = Peak discharge in CFS, subscript indicates recurrence interval, in years
A = Drainage area in square miles
ELEV/1000 = Mean Basin Elevation, in feet above sea level divided by 1000
(LAT - 28/10) = Latitude of the gaged site in decimal degrees minus 28 divided by 10

NOTES:
1. For regions outside shaded areas see Figure 819.2C
2. The equations are applicable to unregulated streams that drain basins of less than about 200 square miles

(3) *National Resources Conservation Service (NRCS) Methods.* The Soil Conservation Service's SCS (former title) National Engineering Handbook, 1972, and their 1975, "Urban Hydrology for Small Watersheds", Technical Release 55 (TR-55), present a graphical method for estimating peak discharge. Most NRCS equations and curves provide results in terms of inches of runoff for unit hydrograph development and are not applicable to the estimation of a peak design discharge unless the design hydrograph is first developed in accordance with prescribed NRCS procedures. NRCS methods and procedures are applicable to drainage areas less than 8 km^2 (800 ha) and result in a design hydrograph and design discharge that are functionally acceptable to form the basis for the design of highway drainage facilities.

819.3 Statistical Methods

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

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

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

Some common types of data groupings are as follows:

- Magnitude
- Time of Occurrence
- Geographic Location

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

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

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

- Mean flow
- Standard deviation
- Coefficient of skew

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

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

(3) *Gumbel Extreme Value Distribution.* The characteristics of the Gumbel extreme value distribution (also known as the double exponential distribution of extreme values) are that the mean flood occurs at the return period

(6) *Spiral Rib Steel.* Galvanized steel spiral rib pipe is fabricated using sheet steel and lock seam fabrication as used for helical corrugated metal pipe. The thickness of metal and zinc coating is identical to that for corrugated pipe. Spiral rib pipe has a lower roughness coefficient (Manning's "n") than corrugated metal pipe.

Aluminized steel spiral rib pipe, type 2 (ASSRP) is available in the same sizes as galvanized steel spiral rib and will support the same fill heights (the aluminizing is simply a replacement coating for zinc galvanizing that allows thinner steel to be placed in certain corrosive environments - See Figure 854.3B for the acceptable pH and resistivity ranges for placement of aluminized steel pipes). Tables 854.3F, G & H give the maximum height of overfill for steel spiral rib pipe constructed under the acceptable methods contained in the Standard Specifications and essentials discussed in Index 829.2.

854.4 Corrugated Aluminum Pipe, Aluminum Spiral Rib Pipe and Pipe Arches

(1) *Hydraulics.* Corrugated aluminum pipe comes in various corrugated profiles. Annular and helical corrugated aluminum pipe configurations are applicable in the situations where velocity reduction is important or if a culvert is being designed with an inlet control condition. Spiral rib pipe, on the other hand, may be more appropriate for use in stormdrain situations or if a culvert is being designed with an outlet control condition.

(2) *Durability.* Aluminum culverts or stormdrains may be specified as an alternate culvert material. When a 50-year maintenance free service life of aluminum pipe is required the pH and minimum resistivity, as determined by California Test Method 643, must be known and the following conditions met:

(a) The pH of the soil, backfill, and effluent is within the range of 5.5 and 8.5, inclusive. Bituminous coatings are not recommended for corrosion protection or abrasion resistance.

(b) The minimum resistivity of the soil, backfill, and effluent is 1500 ohm-centimeters or greater.

(c) Under similar conditions, aluminum culverts will abrade approximately three times faster than steel culverts. Therefore, aluminum culverts are not recommended where abrasive materials are present, and where flow velocities would encourage abrasion to occur. Culvert flow velocities that frequently exceed 1.5 m/s should be carefully evaluated prior to selecting aluminum as an allowable alternate.

(d) Aluminum culverts should not be installed in an environment where other aluminum culverts have exhibited significant distress, such as extensive perforation or loss of invert, for whatever reason, apparent or not.

(e) Aluminum may be considered for side drains in environments having the following parameters:

- When pH is between 5.5 and 8.5 and the minimum resistivity is between 500 and 1500 ohm-cm.
- When pH is between 5.0 and 5.5 or between 8.5 and 9.0 and the minimum resistivity is greater than 1500 ohm-cm.

For these conditions, the METS should be contacted to confirm the advisability of using aluminum on specific projects.

(f) Aluminum must not be used as a section or extension of a culvert containing steel sections.

Figure 854.3B should be used to determine the limitations on the use of corrugated aluminum pipe for various levels of pH and minimum resistivity. The minimum thickness (1.5 mm) of aluminum pipe obtained from the chart only satisfies corrosion requirements. Overfill requirements for minimum metal thickness must also be satisfied. The metal thickness of corrugated aluminum pipe should satisfy both requirements.

(3) *Strength Requirements.* The strength requirements for corrugated aluminum pipe and pipe arches fabricated under the acceptable methods contained in the Standard Specifications, are given in Tables 854.4A, B & C. For aluminum spiral rib pipe, see Tables 854.4D & E.

(a) Design Standards.

- Corrugation Profiles - Corrugated aluminum pipe and pipe arches are available in 68 mm x 13 mm and 75 mm x 25 mm profiles with helical or annular corrugations. Aluminum spiral rib pipe is available in a 19 mm x 19 mm x 190 mm or a 19 mm x 25 mm x 292 mm helical corrugation profile.
- Metal thickness - Corrugated aluminum pipe and pipe arches are available in the thickness as indicated on Tables 854.4A, B & C. Where a maximum overfill is not listed on these tables, the pipe or pipe arch is not normally available in that thickness. Aluminum spiral rib pipe are available in the thickness as indicated on Tables 854.4D & E.
- Height of Fill - The allowable overfill heights for corrugated aluminum pipe and pipe arches for various diameters and metal thickness are shown on Tables 854.4A, B & C. For aluminum spiral rib pipe, overfill heights are shown on Tables 854.4D, & E.

To properly use the above mentioned tables, the designer should be aware of the basic premises on which the tables are based as well as their limitations. (See Index 854.3(2)).

(4) *Shapes.* Corrugated aluminum pipe, aluminum spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. Helical corrugated pipe must be specified if anticipated heights of cover exceed the tabulated values for annular corrugated pipe.

For larger diameters, arch spans, or special shapes, see Index 854.6. Non-standard pipe diameters and arch sizes are also available.

(5) *Invert Protection.* Invert protection of corrugated aluminum is not recommended.

(6) *Spiral Rib Aluminum.* Aluminum spiral rib pipe is similar to spiral rib steel. Figure 854.3B should be used to determine the limitations on the use of spiral rib aluminum pipe for the various levels of pH and minimum resistivity. Tables 854.4D & E give the maximum overfill for aluminum spiral rib pipe constructed under the acceptable methods contained in the Standard Specifications and the essentials discussed in Index 829.2.

854.5 Special Purpose Types

(1) *Smooth Steel.* Smooth steel (welded) pipe can be utilized for drainage facilities under conditions where corrugated metal or concrete pipe will not meet the structural or design service life requirements.

(2) *Composite Steel Spiral Rib Pipe.* Composite steel spiral rib pipe is a smooth interior pipe with efficient hydraulic characteristics. See Table 851.2.

Composite steel spiral rib pipe with its interior polyethylene liner exhibits good abrasion resistance and also resists corrosion from chemicals found in a typical stormdrain or sanitary sewer environment. The exterior of the pipe is protected with a polyethylene film which offers resistance to corrosive backfills. The pipe will meet a 50 year maintenance free service life under most conditions.

(3) *Proprietary Pipe.* See Indexes 110.10 and 601.5(3) for further discussion and guidelines on the use of proprietary items.

854.6 Structural Metal Plate

(1) *Pipe and Arches.* Structural plate pipes and arches are available in steel and aluminum for the diameters and thickness as shown on Tables 854.6A, B, C & D.

- Height of Fill - The allowable overfill heights for plastic pipe for various diameters are shown in Table 854.8.

(b) Basic Premise. To properly use the plastic pipe height of fill table, the designer should be aware of the basic premises on which the table is based as well as their limitations. The design table presupposes:

- That bedding and backfill satisfy the terms of the Standard Specifications, the conditions of cover, and pipe size required by the plans and the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.

854.9 Minimum Height of Cover

Table 854.9 gives the minimum thickness of cover required for design purposes over pipes and pipe arches. For construction purposes, a minimum cover of 150 mm greater than the pavement structural section is desirable for all types of pipe.

Where cover heights above culverts are less than the values shown in Table 854.9, stress reducing slab details available from the HQ's Design drainage detail library at the following web address may be used: http://pd.dot.ca.gov/design/drainage.asp

Where cover heights are less than the values shown in the stress reducing slab details, contact the Office of State Highway Drainage Design or the Underground Structures Branch of the Division of Engineering Services - Office of Design & Technical Services.

Table 854.8

Thermoplastic Pipe Fill Height Tables

High Density Polyethylene (HDPE) Corrugated Pipe

Size (mm)	Maximum Height of Cover (m)
300	9.0
375	9.0
450	9.0
600	9.0
750	9.0
900	9.0
1050	6.0
1200	6.0

High Density Polyethylene (HDPE) Ribbed Pipe

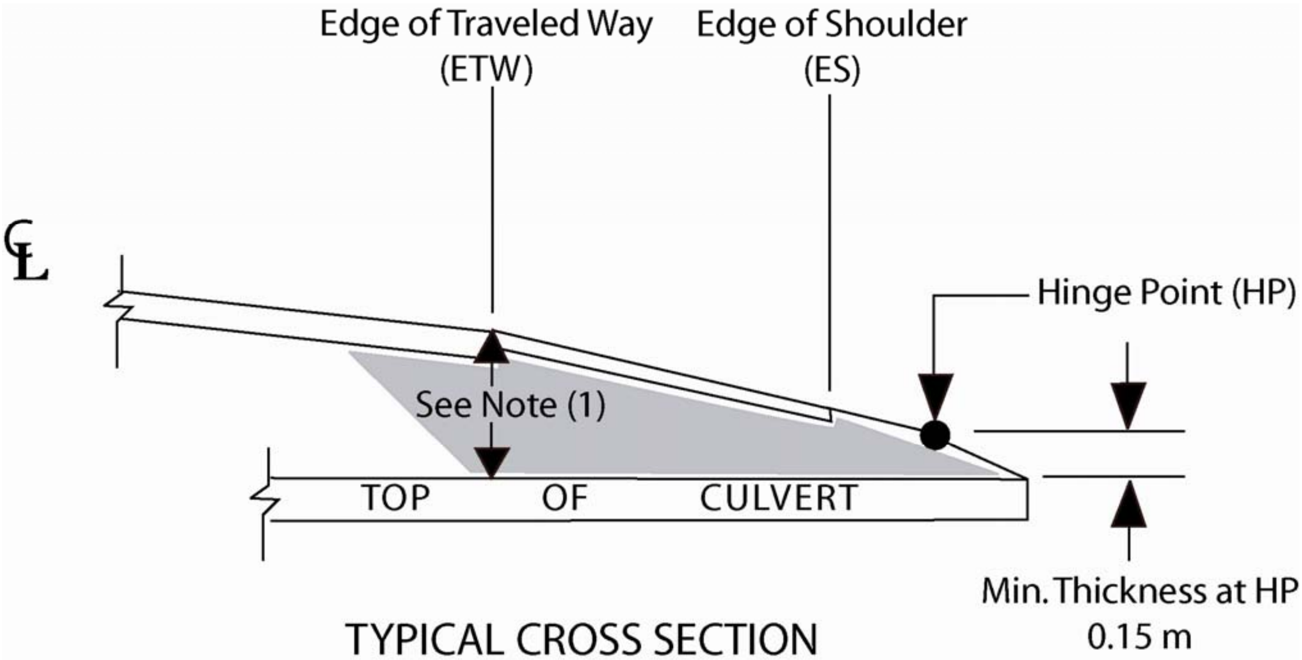
Size (mm)	Maximum Height of Cover (m)
450	7.3
525	7.3
600	7.3

Polyvinyl Chloride (PVC) Ribbed Pipe

Size (mm)	Maximum Height of Cover (m)
450	8.2
525	7.9
600	7.6
675	7.3
750	7.0
900	6.7
1050	6.4
1200	6.1

Table 854.9

Minimum Thickness of Cover for Culverts



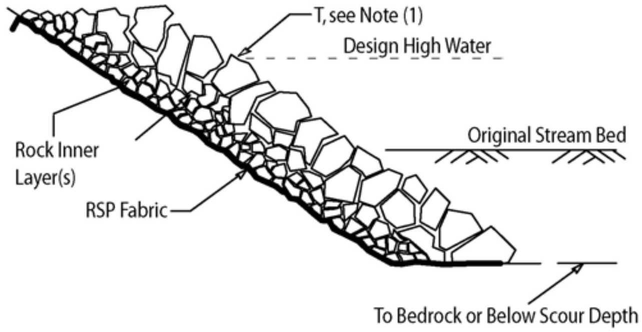
MINIMUM THICKNESS OF COVER AT ETW

SURFACE TYPE	Corrugated metal pipes and pipe-arches	Structural plate pipes and pipe-arches	Reinforced concrete pipes	Plastic pipes	Cast-In-Place concrete pipes (2)
Flexible Pavements or Unpaved	1/5 (dia. or span) or 0.6 m minimum.	1/8 (dia. or span) or 0.6 m minimum	0.6 m minimum	0.6 m minimum	Structural Section plus 0.6 m.
Rigid Pavements	1/5 (dia. or span) or 0.4 m minimum.	1/8 (dia. or span) or 0.4 m minimum	0.3 m minimum	0.6 m minimum	Structural Section plus 0.6 m.

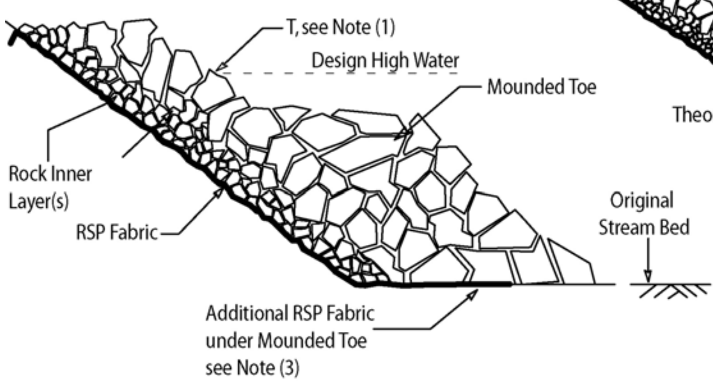
Notes: (1) Minimum thickness of cover is measured at ultimate or future edge of traveled way.
(2) See Index 854.2(1)(d) for necessary approvals prior to placing cast-in-place concrete pipes under the roadway.

Figure 873.3C
Rock Slope Protection

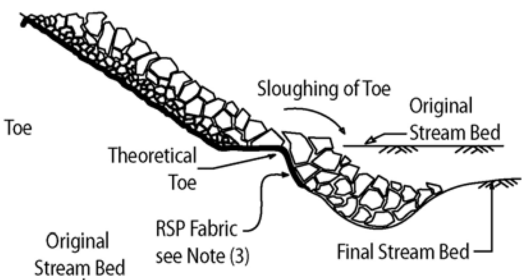
Embedded Toe RSP



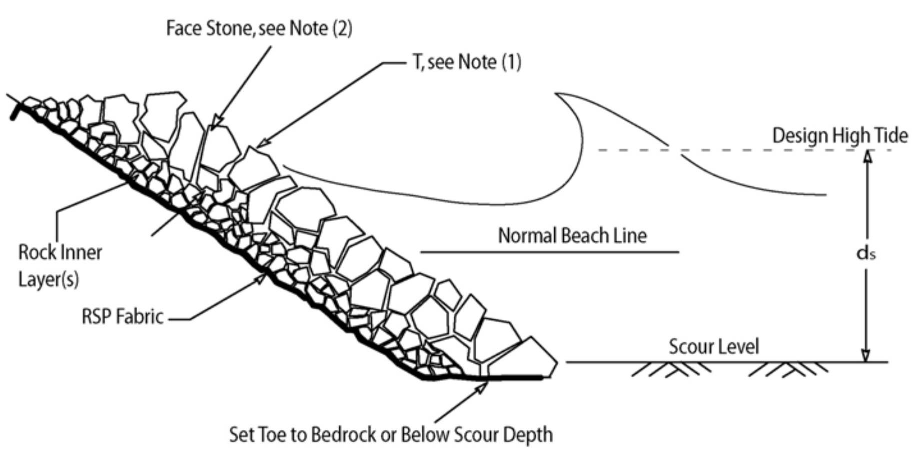
Mounded Toe RSP
(as constructed)



Mounded Toe RSP
(after launching of Mounded Toe)



Shore Protection RSP



Notes:

(1) Thickness "T" from Table 873.3 C.
(2) Face stone is determined from Figure 873.3G.
(3) RSP fabric not to extend more than 20 percent the base width of the Mounded Toe past the Theoretical Toe.

(c) Design Example -- The following example reflects the CABS method for designing RSP as described in Report No. FHWA – CA – TL – 95 – 10, as well as identify some of the considerations and technical principles that the designer must address to complete the installation design. These same considerations and principles apply to concreted RSP as well as RSP placed on beaches and shores (which are covered later), and therefore, separate examples for those designs are not provided. The designer is encouraged to review the entire report referenced above, available on the Division of Design website, for a comprehensive discussion of the basis of the CABS method and RSP design considerations.

The following example assumes that the designer has conducted the appropriate site assessments and resulting calculations to establish average stream velocity, estimated depth of scour, stream alignment (i.e., parallel or impinging flow), length of stream bank to be protected and locations of natural hard points (e.g., rock outcroppings). Field reviews and discussions with maintenance staff familiar with the site are critical to the success of the design.

Given for example:

- Average stream velocity for design event – 4.9 m/s (16 fps)
- Estimated scour depth – 1.7 m
- Length of bank requiring protection – 150 m
- Bank slope – 1V:1.5H
- Specific gravity of rock used for RSP – 2.65 (based on data from local quarry)
- Embankment is on outside of stream bend

1) Calculate minimum rock mass for outer layer:

$$W = \frac{(0.00002)(16 \times 4/3)^6 (2.65)}{(2.65-1)^3 \sin^3 (70-33.69)}$$

$$W = 5{,}350 \text{ lb}$$

$$W = 2.67 \text{ ton} \Rightarrow 2.43 \text{ tonne}$$

NOTES:

a. Equation inputs must be in U.S. Customary (English) Units. Convert calculated rock mass to metric tonnes to continue design.

b. For ease of computation with hand held calculators, cosecant has been converted to 1/sine.)

2) Select gradation for outer layer.

a) From minimum calculated rock mass of 2.43 tonne in the example, select the rock mass from the left-side column tables in Standard Specification Section 72-2.02 that represents the standard rock mass just larger than the calculated mass. For ease, the Standard Specification tables are combined and reprinted in Table 873.3A.

gabions may be more aesthetically acceptable than rock riprap. Where larger stone sizes are not readily available and the flow does not abrade the wire baskets, they may also be more cost effective. However, caution is advised regarding in-stream placement of gabions, and some form of abrasion protection in the form of wooden planks or other facing will typically be necessary, see Figure 873.3E.

Figure 873.3E

Gabion Lined Streambank

Gabion wall with timber facing to protect wires from abrasive flow.

Refer to the draft Gabion Geotechnical Design Bulletin, available at the following Caltrans Intranet site: http://onramp.dot.ca.gov/hq/esc/sd/bridge_design/ers/documents/gabion_dib.pdf for further discussion on the use of gabions for slope protection.

(d) Articulated Precast Concrete. This type of revetment consists of pre-cast concrete blocks which interlock with each other, are attached to each other, or butted together to form a continuous blanket or mat. A number of block designs are commercially available. They differ in shape and method of articulation, but share common features of flexibility and rapid installation. Most provide for establishment of vegetation within the revetment.

The permeable nature of these revetments permits free draining of the embankment and their flexibility allows the mat to adjust to minor changes in bank geometry. Pre-cast concrete block revetments may be economically justified where suitable rock for slope protection is not readily available. They are generally more aesthetically pleasing than other types of revetment, particularly after vegetation has become established.

Individual blocks are commonly joined together with steel cable or synthetic rope, to form articulated block mattresses. Pre-assembled in sections to fit the site, the mattresses can be used on slopes up to 1:2. They are anchored at the top of the revetment to secure the system against slippage.

Pre-cast block revetments that are formed by butting individual blocks end to end, with no physical connection, should not be used on slopes steeper than 1:3. An engineering fabric is normally used on the slope to prevent the erosion of the underlying embankment through the voids in the concrete blocks.

Refer to HEC-11, Design of Riprap Revetment, Section 6.2, and HEC-23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 4, for further discussion on the use of articulated concrete blocks.

(3) Rigid Revetments.

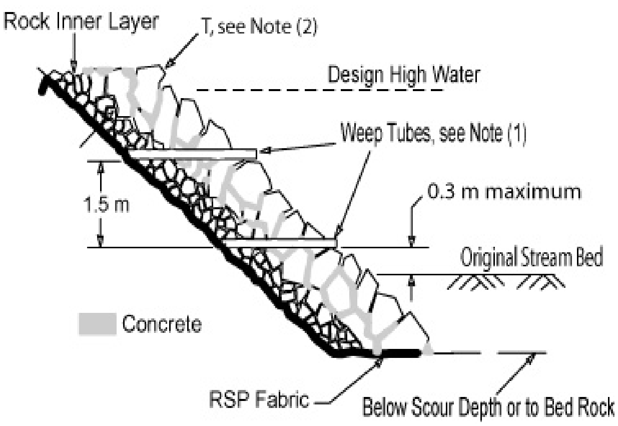
(a) Concreted-Rock Slope Protection.

(1) General Features. This type of revetment consists of rock slope protection with interior voids filled with PCC to form a monolithic armor. A typical section of this type of installation is shown in Figure 873.3F.

It has application in areas where rock of sufficient size for ordinary rock slope protection is not economically available.

Figure 873.3F

Concreted-Rock Slope Protection



Notes:

(1) If needed to relieve hydrostatic pressure.

(2) Refer to Table 873.3 C for section thickness.

Dimensions and details should be modified as required.

(2) Design Concepts. Concreting of RSP is a common practice where availability of large stones is limited, or where there is a need to reduce the total thickness of a RSP revetment. Inclusion of the concrete, and the labor required to place it, makes concreted RSP installations more expensive per unit area than non-concreted installations.

Design procedures for concreted RSP revetments are similar to that of non-concreted RSP. Start by following the design example provided in Index 873.3(2)(a)(2)(c) to select a stable rock size for a non-concreted design based on the site conditions. This non-concreted rock size is divided by a factor of roughly four or five to arrive at the appropriate size outer layer rock for a concreted revetment. The factor is based on observations of previously constructed facilities and represents the typical sized pieces that stay together even after severe cracking (i.e., failed revetments will still usually have segments of four to five rocks holding together). As with the non-concreted design procedures, use the rock size derived from this calculation to enter Table 873.3A (i.e., round up to the next larger rock mass, which will represent the 50-100 percentage larger than gradation range) and then select the appropriate RSP Class. The thickness and rock sizing of the inner layers can be based on the reduced sizing of the outer layer rock. Note that as shown in Figure 873.3F, the inner layers of rock are not concreted.

C

U