CALIFORNIA
DEPARTMENT OF TRANSPORTATION

Manual Change Transmittal

TITLE	APPROVED BY	NO.
HIGHWAY DESIGN MANUAL SEVENTH EDITION – CHANGE 05/20/22	JANICE BENTON, Chief	Date Issued: 05/20/22 Page 1 of 3
SUBJECT AREA Table of Contents; Chapters: 60, 80, 100, 200, 300, 400, 500, 600, 610, 630, 640, 660, 680, 700, 850, 860, 870, 880, and Index	ISSUING UNIT DIVISION OF DESIGN	
SUPERCEDES SEE BELOW FOR SPECIFIC PAGE NUMBERS	DISTRIBUTION **ALL HOLDERS OF THE 7TH EDITION, HIGHWAY DESIGN MANUAL**	

The Table of Contents; Chapters: 60, 80, 100, 200, 300, 400, 500, 600, 610, 630, 640, 660, 680, 700, 850, 860, 870, 880; and the Index of the Seventh Edition, Highway Design Manual (HDM) have been revised. The changes to the HDM are summarized below with change sheets available on the Department Design website at: https://dot.ca.gov/programs/design/manual-highway-design-manual-hdm.
Changes include updates related to curb ramps, temporary barrier with falsework openings, rumble strips, median width, roundabouts, interchange spacing, ramp metering, pavement design, and plastic pipe. Also, included are website updates, clarification language, typographical corrections, reference corrections, and updates to figures and tables.

These changes are effective May 20, 2022 and shall be applied to on-going projects in accordance with HDM Index 82.5 – Effective Date for Implementing Revisions to Design Standards.

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

A summary of the most significant revisions made throughout the manual are as follows:

Index 81.4 **Type of Highway, Page 80-5**
A link is provided for the California Road System map. This map displays the functional classification of roadways for the State, and is approved by the Federal Highway Administration.

Index 105.5	**Curb Ramps, Page 100-13** Clarification is provided when a curb ramp can serve two crossings.
Index 204.8(5) and Table 204.8	**Falsework, Pages 200-33 and 34** K-rail is replaced with temporary barrier. The section and Table have been updated for normal span and minimum falsework depth.
Index 302.1	**Rumble Strips, Page 300-5** This section refers to the Traffic Safety Bulletin 20-07 for placement of rumble strips and rumble stripes in and adjacent to shoulders.
Index 305.1	**Median Width, Page 300-22** Clarification for minimum median width is provided for multilane freeways and expressways.
Index 404.2	**Design Considerations, Page 400-15** A reference and link is provided to the Caltrans Truck Network Maps.
Index 405.10	**Roundabouts, Pages 400-46 to 55** Various updates and clarifications are provided. These include guidance for three design vehicles, pedestrian crossing, landscape buffer/strip, sidewalk, and horizontal clearance.
Index 501.3	**Interchange Spacing, Page 500-1** This update provides a more accurate compliance with the AASHTO publication "A Policy on Design Standards—Interstate System."
Index 504.3(2)	**Ramp Metering, Pages 500-17 to 26** Various updates and clarification are provided.
Chapters 600, 610, 630, 640, 660, 680	**Chapter 600 series** Implementation of new flexible pavement design method called Mechanistic-Empirical Design Method. The method was approved by Pavement Program Steering Committee in 2005. An online computer software (CalME-Software) has also been developed to facilitate the pavement analysis method.

<u>**Index 701.2(5)**</u> **Locked Gates, Pages 700-3 and 4**
Language was updated to clarify when FHWA or District Director's approval is needed, depending on whether the location is on the Interstate or not.

<u>**Chapter 850**</u> **Drainage Facility Material Types, Various pages throughout**
Updates for Polypropylene (PP) as an alternative plastic culvert option to HDPE and PVC.

<u>**Chapter 870**</u> **Bank Protection – Erosion Control, Various pages throughout**
Included new plant tube language, new geomorphology clarification, and other minor edits.

<u>**Index 883.3**</u> **Armor Protection, Pages 880-25 to 28**
Removed references to crib wall.

Enclosures are included with this manual change transmittal of the total replacement pages.

Table of Contents
Figures and Tables

theatres, entertainment and convention venues, government buildings, and little or no industry because of the high value of land. Historic sections may be referred to as “old town.”

(5) *Condemnation.* The process by which property is acquired for public purposes through legal proceedings under power of eminent domain.

(6) *Control of Access.* The condition where the right of owners or occupants of abutting land or other persons to access in connection with a highway is fully or partially controlled by public authority.

(7) *Easement.* A right to use or control the property of another for designated purposes.

(8) *Eminent Domain.* The power to take private property for public use without the owner's consent upon payment of just compensation.

(9) *Encroachment.* In terms of exceptions and permits, includes, but is not limited to, any structure, object, or activity of any kind or character which is within the State right of way, but it is not a part of the State facility or serving a transportation need.

(10) *Inverse Condemnation.* The legal process which may be initiated by a property owner to compel the payment of just compensation, where the property has been taken for or damaged by a public purpose.

(11) *Negotiation.* The process by which property is sought to be acquired for project purposes through mutual agreement upon the terms for transfer of such property.

(12) *Partial Acquisition.* The acquisition of a portion of a parcel of property.

(13) *Relinquishment.* A transfer of the State's right, title, and interest in and to a highway, or portion thereof, to a city or county.

(14) *Right of Access.* The right of an abutting land owner for entrance to or exit from a public road.

(15) *Severance Damages.* Loss in value of the remainder of a parcel which may result from a partial taking of real property and/or from the project.

(16) *Vacation.* The reversion of title to the owner of the underlying fee where an easement for highway purposes is no longer needed.

62.7 Pavement

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

(1) *Asphalt Concrete.* See Hot Mix Asphalt (HMA).

(2) *Asphalt Rubber.* A blend of asphalt binder, reclaimed tire rubber, and certain additives in which the rubber component is at least 15 percent by weight of the total blend and has reacted in the hot asphalt binder sufficiently to cause swelling of the rubber particles.

(3) *Asphalt Treated Permeable Base (ATPB).* A highly permeable open-graded mixture of crushed coarse aggregate and asphalt binder placed as the base layer to assure adequate drainage of the structural section, as well as structural support.

(4) *Base.* A layer of selected, processed, and/or treated aggregate material that is placed immediately below the surface course. It provides additional load distribution and contributes to drainage and frost resistance.

(5) *Basement Soil/Material.* See Subgrade.

(6) *Borrow.* Natural soil obtained from sources outside the roadway prism to make up a deficiency in excavation quantities.

(7) *California R-Value.* A measure of resistance to deformation of the soils under saturated conditions and traffic loading as determined by the stabilometer test (CT301). The California R-value, also referred to as R-value, measures the supporting strength of the subgrade and subsequent layers used in the pavement structure. For additional information, see Topic 614.

(8) *Capital Preventive Maintenance.* Typically, Capital Preventive Maintenance (CAPM) consists of work performed to preserve the existing pavement structure utilizing strategies that preserve or extend pavement service life. The CAPM program is divided into pavement preservation and pavement rehabilitation. For further discussion see Topic 603.

(9) *Cement Treated Permeable Base (CTPB).* A highly permeable open-graded mixture of coarse aggregate, portland cement, and water placed as the base layer to provide adequate drainage of the structural section, as well as structural support.

(10) *Composite Pavement.* These are pavements comprised of both rigid and flexible layers. Currently, for purposes of the procedures in this manual, only flexible over rigid composite pavements are considered composite pavements.

(11) *Crack.* Separation of the pavement material due to thermal and moisture variations, consolidation, vehicular loading, or reflections from an underlying pavement joint or separation.

(12) *Crack, Seat, and Overlay (CSO).* A rehabilitation strategy for rigid pavements. CSO practice requires the contractor to crack and seat the rigid pavement slabs, and place a flexible overlay with a pavement reinforcing fabric (PRF) interlayer.

(13) *Crumb Rubber Modifier (CRM).* Scrap rubber produced from scrap tire rubber and other components, if required, and processed for use in wet or dry process modification of asphalt paving.

(14) *Deflection.* The downward vertical movement of a pavement surface due to the application of a load to the surface.

(15) *Dense Graded Asphalt Concrete (DGAC).* See Hot Mix Asphalt (HMA).

(16) *Depression.* Localized low areas of limited size that may or may not be accompanied by cracking.

(17) *Dowel Bar.* A load transfer device in a rigid slab usually consisting of a plain round steel bar.

(18) *Edge Drain System.* A drainage system, consisting of a slotted plastic collector pipe encapsulated in treated permeable material and a filter fabric barrier, with unslotted plastic pipe vents, outlets, and cleanouts, designed to drain both rigid and flexible pavement structures.

(19) *Embankment.* A prism of earth that is constructed from excavated or borrowed natural soil and/or rock, extending from original ground to the grading plane, and designed to provide a stable support for the pavement structure.

(20) *Equivalent Single Axle Loads (ESAL's).* The number of 18-kip standard single axle load repetitions that would have the same damage effect to the pavement as an axle of a specified magnitude and configuration. See Index 613.2 for additional information.

(21) *Flexible Pavement.* Pavements engineered to transmit and distribute vehicle loads to the underlying layers. The highest quality layer is the surface course (generally asphalt binder mixes) which may or may not incorporate underlying layers of base and subbase. These types of pavements are called "flexible" because the total pavement structure bends or flexes to accommodate deflection bending under vehicle loads. For further discussion, see Chapter 630.

(22) *Grading Plane.* The surface of the basement material upon which the lowest layer of subbase, base, pavement surfacing, or other specified layer, is placed.

(23) *Gravel Factor (G_f).* Refers to the relative strength of a given material compared to a standard gravel subbase material. The cohesiometer values were used to establish the G_f currently used by Caltrans.

(24) *Hot Mix Asphalt (HMA).* Formerly known as asphalt concrete (AC), HMA is a graded asphalt concrete mixture (aggregate and asphalt binder) containing a small percentage of voids which is used primarily as a surface course to provide the structural strength needed to distribute loads to underlying layers of the pavement structure.

(25) *Hot Recycled Asphalt (HRA).* The use of reclaimed flexible pavement which is combined with virgin aggregates, asphalt, and sometimes rejuvenating agents at a central hot-mix plant and placed in the pavement structure in lieu of using all new materials.

(26) *Joint Seals.* Pourable, extrudable or premolded materials that are placed primarily in transverse and longitudinal joints in concrete pavement to deter the entry of water and incompressible materials (such as sand that is broadcast in freeze-thaw areas to improve skid resistance).

(27) *Lean Concrete Base.* Mixture of aggregate, portland cement, water, and optional admixtures, primarily used as a base for portland cement concrete pavement.

(28) *Longitudinal Joint.* A joint normally placed between roadway lanes in rigid pavements to control longitudinal cracking; and the joint between the traveled way and the shoulder.

(29) *Maintenance.* The preservation of the entire roadway, including pavement structure, shoulders, roadsides, structures, and such traffic control devices as are necessary for its safe and efficient utilization.

(30) *Open Graded Asphalt Concrete (OGAC).* See Open Graded Friction Course (OGFC).

(31) *Open Graded Friction Course (OGFC).* Formerly known as open graded asphalt concrete (OGAC), OGFC is a wearing course mix consisting of asphalt binder and aggregate with relatively uniform grading and little or no fine aggregate and mineral filler. OGFC is designed to have a large number of void spaces in the compacted mix as compared to hot mix asphalt. For further discussion, see Topic 631.

(32) *Overlay.* An overlay is a layer, usually hot mix asphalt, placed on existing flexible or rigid pavement to restore ride quality, to increase structural strength (load carrying capacity), and to extend the service life.

(33) *Pavement.* The planned, engineered system of layers of specified materials (typically consisting of surface course, base, and subbase) placed over the subgrade soil to support the cumulative vehicle loading anticipated during the design life of the pavement. The pavement is also referred to as the pavement structure and has been referred to as pavement structural section.

(34) *Pavement Design Life.* Also referred to as performance period, pavement design life is the period of time that a newly constructed or rehabilitated pavement is engineered to perform before reaching a condition that requires CAPM, (see Index 603.3). The selected pavement

design life varies depending on the characteristics of the highway facility, the objective of the project, and projected vehicle volume and loading.

(35) *Pavement Drainage System.* A drainage system used for both asphalt and rigid pavements consisting of a treated permeable base layer and a collector system which includes a slotted plastic pipe encapsulated in treated permeable material and a filter fabric barrier with unslotted plastic pipe as vents, outlets and cleanouts to rapidly drain the pavement structure. For further discussion, see Chapter 650.

(36) *Pavement Preservation.* Work done, either by contract or by State forces to preserve the ride quality, safety characteristics, functional serviceability and structural integrity of roadway facilities on the State highway system. For further discussion, see Topic 603.

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

(38) *Pavement Structure.* See Pavement.

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

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

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

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

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

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

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

(46) *Rubberized Hot Mix Asphalt (RHMA).* Formerly known as rubberized asphalt concrete (RAC). RHMA is a material produced for hot mix applications by mixing either asphalt rubber

(b) High Density Urban Main Streets.

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

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

81.4 Type of Highway

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

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

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

The California Road System (CRS) maps are the official functional classification maps approved by FHWA. These maps show functional classification of roads. See the link at https://caltrans.maps.arcgis.com/apps/webappviewer/index.html?id=026e830c914c495797c969a3e5668538.

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

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

81.5 Access Control

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

81.6 Design Standards and Highway Context

The design standards were initially established to increase highway mobility and development, promoting a State transportation system that operated at selected levels of service consistent with projected traffic volumes and highway classification. Design standards revolved around FHWA's controlling criteria, evolving over time to more fully consider adjacent community values, local decisions making, and area context.

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

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

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

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

Topic 82 – Application of Standards

82.1 Highway Design Manual Standards

(1) *General.* The highway design criteria and policies in this manual provide a guide for the engineer to exercise sound judgment in applying standards, consistent with the above Project Development philosophy, in the design of projects. This guidance allows for

82.2 Approvals for Nonstandard Design

(1) *Boldface Standards*. Design features or elements which deviate from standards indicated in boldface type require the approval of the Chief, Division of Design. This approval authority has been delegated to the District Directors for projects on conventional highways and expressways, and for certain other facilities in accordance with the current District Design Delegation Agreement. Approval authority for design standards indicated in boldface type on all other facilities has been delegated to the Project Delivery Coordinators except as noted in Table 82.1A where: (a) the standard has been delegated to the District Director, (b) the standards in Chapters 600 through 680 requires the approval of the State Pavement Engineer, and (c) specifically delegated to the District Director per the current District Design Delegation Agreements and may involve coordination with the Project Delivery Coordinator. See the HQ Division of Design website for the most current District Design Delegation Agreements.

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

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

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

FHWA approval is not required for deviations from "Caltrans-only" standards. Table 82.1A identifies these "Caltrans-only" standards. Where FHWA approval of a deviation from a design standard is required, only cite the standards that are identified by the FHWA as ten controlling criteria, see Index 82.1(3).

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

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

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

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

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

82.3 FHWA and AASHTO Standards and Policies

The standards in this manual generally conform to the standards and policies set forth in the AASHTO publications, "A Policy on Geometric Design of Highways and Streets" (2018) and "A Policy on Design Standards-Interstate System" (2016). A third AASHTO publication, the latest edition of the "Roadside Design Guide", focuses on creating safer roadsides. These three documents, along with other AASHTO and FHWA publications cited in 23 CFR Ch 1, Part 625, Appendix A, contain most of the current AASHTO policies and standards, and are approved references to be used in conjunction with this manual.

AASHTO policies and standards, which are established as nationwide standards, do not always satisfy California conditions. When standards differ, the instructions in this manual govern, except when necessary for FHWA project approval (Index 108.7, Coordination with the FHWA).

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

82.4 Mandatory Procedural Requirements

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

82.5 Effective Date for Implementing Revisions to Design Standards

Revisions to design standards will be issued with a stated effective date. It is understood that all projects will be designed to current standards unless a design decision has been approved in accordance with Index 82.2 or otherwise noted by separate Design Memorandum.

On projects where the project development process has started, the following conditions on the effective date of the new or revised standards will be applied: For all projects where the PS&E has not been finalized, the new or revised design standards shall be incorporated unless this would impose a significant delay in the project schedule or a significant increase in the project engineering or construction costs. The Project Delivery Coordinator or individual delegated authority must make the final determination on whether to apply the new or previous design standards on a project-by-project basis for roadway features.

- For all projects where the PS&E has been submitted to Headquarters Office Engineer for advertising or the project is under construction, the new or revised standards will be incorporated only if they are identified in the Change Transmittal as requiring special implementation.

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

82.6 Design Information Bulletins and Other Caltrans Publications

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

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

82.7 Traffic Engineering

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

The infrastructure within a highway or freeway corridor, segment, intersection or interchange is not "complete" for drivers, bicyclists and pedestrians unless it includes the appropriate traffic control devices; traffic safety systems; operational features or strategies; and traffic management elements and or systems. The presence or absence of these traffic elements and systems can have a profound effect on safety and operational performance. As such, they are commonly employed to remediate performance deficiencies and to optimize the overall performance of the "built" highway system. For additional information visit the Division of Traffic Operations website at http://www.dot.ca.gov/trafficops/.

Table 82.1A

Boldface Standards

CHAPTER 100	BASIC DESIGN POLICIES
Topic 101	Design Speed
Index 101.1	Technical Reductions of Design Speed
101.1	Selection of Design Speed - Local Facilities
101.1	Selection of Design Speed - Local Facilities - with Connections to State Facilities
101.2	Design Speed Standards
Topic 104	Control of Access
Index 104.4	Protection of Access Rights(1)
CHAPTER 200	GEOMETRIC DESIGN AND STRUCTURE STANDARDS
Topic 201	Sight Distance
Index 201.1	Stopping Sight Distance Standards
Topic 202	Superelevation
Index 202.2	Standards for Superelevation
202.7	Superelevation on City Streets and County Roads
Topic 203	Horizontal Alignment
Index 203.1	Horizontal Alignment - Local Facilities
203.1	Horizontal Alignment and Stopping Sight Distance
203.2	Standards for Curvature – Minimum Radius
203.2	Standards for Curvature – Lateral Clearance
Topic 204	Grade
Index 204.1	Standards for Grade - Local Facilities
204.3	Standards for Grade

Design exception approval of Boldface Standards for nonfreeway facilities, including local streets and roads at interchanges, has been delegated to the Districts. In addition, some District delegations included Boldface Standards applicable to freeways. See your District Design Delegation Agreement for specific delegation.

(1) Caltrans-only Boldface Standard.

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

Table 82.1A

Boldface Standards (Cont.)

204.8	Vertical Falsework Clearances(1)
Topic 205	Road Connections and Driveways
Index 205.1	Sight Distance Requirements for Access Openings on Expressways
Topic 208	Bridges, Grade Separation Structures, and Structure Approach Embankment
Index 208.1	Bridge Width(1)
208.4	Bridge Sidewalk (Width)(1)
208.10	Barriers on Structures with Sidewalks(1)
208.10	Bridge Approach Railings(1)
CHAPTER 300	GEOMETRIC CROSS SECTION
Topic 301	Traveled Way Standards
Index 301.1	Lane Width
301.2	Class II Bikeway Lane Width(1)
301.3	Cross Slopes – New Construction
301.3	Cross Slopes – Resurfacing or widening
301.3	Cross Slopes – Unpaved Roadway
301.3	Algebraic Differences in Cross Slopes
Topic 302	Shoulder Standards
Index 302.1	Shoulder Width
302.2	Shoulder Cross Slopes -Bridge
302.2	Shoulder Cross Slopes – Left
302.2	Shoulder Cross Slopes – Paved Median
302.2	Shoulder Cross Slopes - Right
Topic 305	Median Standards

Design exception approval of Boldface Standards for nonfreeway facilities, including local streets and roads at interchanges, has been delegated to the Districts. In addition, some District delegations included Boldface Standards applicable to freeways. See your District Design Delegation Agreement for specific delegation.

(1) Caltrans-only Boldface Standard.

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

Table 82.1A

Boldface Standards (Cont.)

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

Design exception approval of Boldface Standards for nonfreeway facilities, including local streets and roads at interchanges, has been delegated to the Districts. In addition, some District delegations included Boldface Standards applicable to freeways. See your District Design Delegation Agreement for specific delegation.

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Table 82.1A

Boldface Standards (Cont.)

309.3	Horizontal Tunnel Clearances(1)
309.3	Vertical Tunnel Clearances
309.4	Lateral Clearance for Elevated Structures(1)
309.5	Structures Across or Adjacent to Railroads - Vertical Clearance
Topic 310	Frontage Roads
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Index 405.2	Left-turn Channelization - Lane Width
405.2	Left-turn Channelization - Lane Width – Restricted Urban
405.2	Two-way Left-turn Lane Width
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CHAPTER 500	TRAFFIC INTERCHANGES
Topic 501	General
Index 501.3	Interchange Spacing(1)
Topic 502	Interchange Types
Index 502.2	Isolated Off-Ramps and Partial Interchanges(1)
502.3	Route Continuity(1)
Topic 504	Interchange Design Standards
Index 504.2	Location of Freeway Entrances & Exits(1)
504.2	Ramp Deceleration Lane and "DL" Distance(1)
504.3	Ramp Lane Width

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

Table 82.1A

Boldface Standards (Cont.)

504.3	Ramp Shoulder Width
504.3	Ramp Lane Drop Taper Past the Limit Line(1)
504.3	Metered Multi-Lane Ramp Lane Drop Taper Past the Limit Line(1)
504.3	Ramp Meters on Connector Ramps(1)
504.3	Metered Connector Lane Drop(1)
504.3	Distance Between Ramp Intersection and Local Road Intersection(1)
504.4	Freeway-to-freeway Connections – Shoulder Width – 1 and 2-Lane
504.4	Freeway-to-freeway Connections – Shoulder Width – 3-Lane
504.7	Minimum Entrance Ramp-to-Exit Ramp Spacing(1)
504.8	Access Control along Ramps(1)
504.8	Access Control at Ramp Terminal(1)
504.8	Access Rights Opposite Ramp Terminals(1)
CHAPTER 610	PAVEMENT ENGINEERING CONSIDERATIONS
Topic 612	Pavement Design Life
Index 612.2	New Construction and Reconstruction(1), (2)
612.3	Widening(1), (2)
612.5	Roadway Rehabilitation(1), (2)
Topic 613	Traffic Considerations
Index 613.4	Specific Traffic Loading Considerations(1), (2)
CHAPTER 620	RIGID PAVEMENT
Topic 622	Engineering Requirements

Design exception approval of Boldface Standards for nonfreeway facilities, including local streets and roads at interchanges, has been delegated to the Districts. In addition, some District delegations included Boldface Standards applicable to freeways. See your District Design Delegation Agreement for specific delegation.

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

Table 82.1A

Boldface Standards (Cont.)

Index 622.5	Transition Panels, Terminal Joints and End Anchors(1), (2)
Index 622.7	Dowel Bars and Tie Bars(1) (2)
Topic 625	Engineering Procedures for Pavement Rehabilitation
Index 625.2	Rigid Pavement Rehabilitation Strategies(1), (2)
Topic 626	Other Considerations
Index 626.2	Shoulder(1), (2)
626.2	Tied Rigid Shoulders or Widened Slab Standards(1), (2)
626.2	Tied Rigid Shoulders or Widened Slab at Ramps and Gore Standard(1), (2)
CHAPTER 630	FLEXIBLE PAVEMENT
Topic 635	Engineering Procedures for Flexible Pavement Rehabilitation
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CHAPTER 700	MISCELLANEOUS STANDARDS
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CHAPTER 900	LANDSCAPE ARCHITECTURE
Topic 904	Planting Design
Index 904.9	Plant Establishment
Topic 905	Irrigation Design
Index 905.2	Water Supply

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

Table 82.1A

Boldface Standards (Cont.)

Topic 912	Roadside Site Design
Index 912.1	Freeway Ramp Design
Topic 913	Safety Roadside Rest Areas
Index 913.5	Public Pay Telephone
CHAPTER 1000	BICYCLE TRANSPORTATION DESIGN
Topic 1003	Design Criteria
Index 1003.1	Class I Bikeway Widths(1)
1003.1	Class I Bikeway Shoulder Width(1)
1003.1	Class I Bikeway Horizontal Clearance(1)
1003.1	Class I Bikeway Structure Width(1)
1003.1	Class I Bikeway Vertical Clearance(1)
1003.1	Class I Bikeway Minimum Separation From Edge of Traveled Way(1)
1003.1	Physical Barriers Adjacent to Class I Bikeways(1)
1003.1	Class I Bikeway in Freeway Medians(1)
1003.1	Class I Bikeway Design Speeds(1)
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1003.1	Bikeway Shoulder Slope(1)
1003.1	Obstacle Posts or Bollards in Bicycle Paths(1)
CHAPTER 1100	HIGHWAY TRAFFIC NOISE ABATEMENT
Topic 1102	Design Criteria
Index 1102.2	Horizontal Clearance to Noise Barrier(1)
1102.2	Noise Barrier on Safety Shape Concrete Barrier(1)

Design exception approval of Boldface Standards for nonfreeway facilities, including local streets and roads at interchanges, has been delegated to the Districts. In addition, some District delegations included Boldface Standards applicable to freeways. See your District Design Delegation Agreement for specific delegation.

(1) Caltrans-only Boldface Standard.

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

Table 82.1B

Underlined Standards

CHAPTER 100	BASIC DESIGN POLICIES
Topic 101	Design Speed
Index 101.1	Selection of Design Speed – Local Facilities
101.1	Selection of Design Speed – Local Facilities – with Connections to State Facilities
101.2	Design Speed Standards
Topic 104	Control of Access
Index 104.5	Relation of Access Opening to Median Opening
Topic 105	Pedestrian Facilities
Index 105.2	Minimum Sidewalk Width – Next to a Building
105.2	Minimum Sidewalk Width – Not Next to a Building
105.5	Curb Ramp for each Crossing
Topic 107	Roadside Installations
Index 107.1	Standards for Roadway Connections
107.1	Number of Exits and Entrances Allowed at Roadway Connections
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Topic 201	Sight Distance
Index 201.3	Stopping Sight Distance on Sustained Grades
201.7	Decision Sight Distance
Topic 202	Superelevation
Index 202.2	Superelevation on Same Plane for Rural Two-lane Roads
202.5	Superelevation Transition
202.5	Superelevation Runoff
202.5	Superelevation in Restrictive Situations
202.6	Superelevation of Compound Curves
202.7	Superelevation on City Streets and County Roads
Topic 203	Horizontal Alignment
Index 203.1	Horizontal Alignment – Local Facilities
203.3	Alignment Consistency and Design Speed

Table 82.1B

Underlined Standards (Cont.)

203.5	Compound Curves
203.5	Compound Curves on One-Way Roads
203.6	Reversing Curves – Transition Length
203.6	Reversing Curves – Transition Rate
Topic 204	Grade
Index 204.1	Standards for Grade – Local Facilities
204.3	Standards for Grade
204.3	Ramp Grades
204.4	Vertical Curves – 2 Percent and Greater
204.4	Vertical Curves – Less Than 2 Percent
204.5	Decision Sight Distance at Climbing Lane Drops
204.6	Horizontal and Vertical Curves Consistency in Mountainous or Rolling Terrain
Topic 205	Road Connections and Driveways
Index 205.1	Access Opening Spacing on Expressways
205.1	Access Opening Spacing on Expressways – Location
Topic 206	Pavement Transitions
Index 206.3	Lane Drop Transitions
206.3	Lane Width Reductions
Topic 208	Bridges, Grade Separation Structures, and Structure Approach Embankment
Index 208.3	Decking of Bridge Medians
208.6	Minimum width of Walkway of Pedestrian Overcrossings
208.6	Minimum Vertical Clearance of Pedestrian Undercrossings
208.6	Class I Bikeways Exclusive Use
208.10	Protective Screening on Overcrossings
208.10	Bicycle Railing Locations
Topic 210	Earth Retaining Systems

Table 82.1B

Underlined Standards (Cont.)

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Topic 301	Traveled Way Standards
Index 301.2	Class II Bikeway Lane Width Adjacent to On-Street Parking,
301.2	Class II Bikeway with Posted Speeds Greater Than 40 Miles Per Hour
301.3	Algebraic Differences of Cross Slopes at Various Locations
Topic 303	Curbs, Dikes, and Side Gutters
303.1	Use of Curb with Posted Speeds of 40 mph and Greater
303.3	Dike Selection
303.4	Bulbout Design
Topic 304	Side Slopes
Index 304.1	Side Slopes 4:1 or Flatter
Topic 305	Median Standards
Index 305.1	Median Width Freeways and Expressways – Urban
305.1	Median Width Freeways and Expressways – Rural
305.1	Median Width Conventional Highways – Urban and Rural Main Streets
305.1	Median Width Conventional Highways – Climbing or Passing Lanes
305.2	Median Cross Slopes
Topic 309	Clearances
Index 309.1	Clear Recovery Zone – 4:1 or Flatter Apply on All Highways
309.1	Clear Recovery Zone – Necessary Highway Features
309.1	Existing Above-Ground Utilities and Existing Large Trees
309.1	Clear Recovery Zone – Discretionary Fixed Objects
309.1	Conventional Highways with Curbs Typically in Urban Areas
309.1	Areas without Curbs to Barriers at Retaining, Pier, or Abutment Walls
309.1	High Speed Rail Clearance
309.5	Structures Across or Adjacent to Railroads – Vertical Clearance

Table 82.1B

Underlined Standards (Cont.)

Topic 310	Frontage Roads
Index 310.2	Outer Separation – Urban and Mountainous Areas
310.2	Outer Separation – Rural Areas
CHAPTER 400	INTERSECTIONS AT GRADE
Topic 403	Principles of Channelization
Index 403.3	Angle of Intersection
403.6	Optional Right-Turn Lanes
403.6	Right-Turn-Only Lane and Bike Use
Topic 404	Design Vehicles and Related Definitions
Index 404.4	STAA Design Vehicles on the National Network, Terminal Access, California Legal, and Advisory routes
404.4	California Legal Design Vehicle Accommodation
404.4	45-Foot Bus and Motorhome Design Vehicle
Topic 405	Intersection Design Standards
Index 405.1	Corner Sight Distance – No Sight Obstruction in Clear Sight Triangle
405.1	Corner Sight Distance – Driver Set Back
405.1	Corner Sight Distance –Minimum Corner Sight Distance and Table
405.1	Corner Sight Distance at Signalized Public Road Intersections
405.1	Corner Sight Distance at Private Road Intersections
405.1	Decision Sight Distance at Intersections
405.3	Curve Radius for Free Right-Turn with Pedestrian Crossing
405.4	Pedestrian Refuge by Area Place Type
405.5	Emergency Openings and Sight Distance
405.5	Median Opening Locations
405.10	Entry Speeds – Single and Multilane Roundabouts
405.10	Pedestrian Crossing Width
405.10	Landscape Buffer/Strip Width
405.10	Sidewalk and Sidewalk Width
405.10	Horizontal Clearance Width

Table 82.1B

Underlined Standards (Cont.)

CHAPTER 500	TRAFFIC INTERCHANGES
Topic 504	Interchange Design Standards
Index 504.2	Ramp Entrance and Exit Standards
504.2	Collector-Distributor Deceleration Lane and “DL” Distance
504.2	Paved Width at Gore
504.2	Contrasting Surface Treatment
504.2	Auxiliary Lanes
504.2	Freeway Exit Nose Design Speed
504.2	Decision Sight Distance at Exits and Branch Connections
504.2	Design Speed and Alignment Consistency at Inlet Nose
504.2	Freeway Ramp Profile Grades
504.2	Differences in Pavement Cross Slopes at Freeway Entrances and Exits
504.2	Vertical Curves Beyond Freeway Exit Nose
504.2	Crest Vertical Curves at Freeway Exit Terminal
504.2	Sag Vertical Curves at Freeway Exit Terminal
504.2	Ascending Entrance Ramps with Sustained Upgrades
504.3	Ramp Terminus Design Speed
504.3	Ramp Lane Drop Taper At 6-foot Separation Point
504.3	Ramp Lane Drop Location
504.3	Metered Entrance Ramps (1 GP + 1 HOV Preferential Lane) Auxiliary Lane
504.3	Metered Entrance Ramps (1 GP + 1 HOV Preferential Lane) Auxiliary Lane on Sustained Grades and Certain Truck Volumes
504.3	HOV Preferential Lane Restrictive Condition Auxiliary Lane
504.3	Metered Multi-Lane Entrance Ramps Lane Drop
504.3	Metered Multi-Lane Entrance Ramps Auxiliary Lane
504.3	Metered Multi-Lane Entrance Ramps Auxiliary Lane on Sustained Grades and Certain Truck Volumes

Table 82.1B

Underlined Standards (Cont.)

504.3	Metered Freeway-to-Freeway Connector Lane Drops
504.3	Ramp Terminals and Grade
504.3	Ramp Terminals and Sight Distance
504.3	Distance between Ramp Intersection and Local Road Intersection
504.3	Entrance Ramp Lane Drop
504.3	Single-Lane Ramp Widening for Passing
504.3	Two-lane Exit Ramps
504.3	Two-lane Exit Ramps and Auxiliary Lanes
504.3	Distance Between Successive On-ramps
504.3	Distance Between Successive Exits
504.4	Freeway-to-freeway Connections Design Speed
504.4	Profile Grades on Freeway-to-freeway Connectors
504.4	Single-lane Freeway-to-freeway Connector Design
504.4	Single-lane Connector Widening for Passing
504.4	Volumes Requiring Branch Connectors
504.4	Merging Branch Connector Design
504.4	Diverging Branch Connector Design
504.4	Merging Branch Connector Auxiliary Lanes
504.4	Diverging Branch Connector Auxiliary Lanes
504.4	Freeway-to-freeway Connector Lane Drop Taper
504.6	Mainline Lane Reduction at Interchanges
504.8	Access Control at Ramp Terminal
CHAPTER 610	PAVEMENT ENGINEERING CONSIDERATIONS
Topic 612	Pavement Design Life
Index 612.6	Temporary Pavements and Detours

Table 82.1B

Underlined Standards (Cont.)

CHAPTER 620	RIGID PAVEMENT
Topic 625	Engineering Procedures for Pavement Rehabilitation
Index 625.2	Rigid Pavement Rehabilitation Strategies
CHAPTER 640	COMPOSITE PAVEMENTS
Topic 645	Engineering Procedures for Pavement Rehabilitation
Index 645.1	General Considerations
CHAPTER 700	MISCELLANEOUS STANDARDS
Topic 701	Fences
Index 701.2	Fences on Freeways and Expressways
CHAPTER 900	LANDSCAPE ARCHITECTURE
Topic 904	Locating Plants
Index 904.4	Median Planting on freeways
904.5	Minimum Tree Setback
904.5	Large trees on freeway and expressway medians
Table 904.5	Large Tree Setback Requirements on Conventional Highways
904.9	Plant Establishment Period
Topic 905	Irrigation Design
Index 905.4	Irrigation Controller
CHAPTER 1000	BICYCLE TRANSPORTATION DESIGN
Topic 1003	Bikeway Design Criteria
Index 1003.1	Class I Bikeway Horizontal Clearance
1003.1	Class I Bikeway in State Highway or Local Road Medians

Table 82.1C

Decision Requiring Other Approvals

CHAPTER 100	BASIC DESIGN POLICIES
Topic 103	Design Designation
Index 103.2	Design Period
Topic 108	Coordination With Other Agencies
Index 108.2	Transit Loading Facilities – Location
108.2	Transit Loading Facilities - ADA
108.3	Rail Crossings*
108.3	Parallel Rail Facilities*
108.5	Bus Rapid Transit – Location and ADA
108.7	Coordination With the FHWA - Approvals
Topic 110	Special Considerations
Index 110.1	Overload Category
110.8	Safety Review Items and Employee Exposure
110.10	Proprietary Items
110.10	Proprietary Items – On Structure
110.10	Proprietary Items – National Highway System
Topic 111	Material Sites and Disposal Sites
Index 111.1	Mandatory Material Sites on Federal-aid Projects
111.6	Mandatory Material Sites and Disposal Sites on Federal-aid Projects
Topic 116	Bicyclists and Pedestrians on Freeway
Index 116	Bicycles and Pedestrians on Freeways
CHAPTER 200	GEOMETRIC DESIGN AND STRUCTURE STANDARDS
Topic 204	Grade

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

Table 82.1C

Decision Requiring Other Approvals (Cont.)

Index 204.8	Grade Line of Structures – Temporary Vertical Clearances
Topic 205	Road Connections and Driveways
Index 205.1	Conversion of a Private Opening
Topic 208.10	Bridge Barriers and Railing
Index 208.10	Barrier Separation and Bridge Rail Selection
208.10	Concrete Barrier Type 80
208.10	Concrete Barrier Type 80SW
208.11	Deviations from Foundation and Embankment Recommendations
210.4	Cost Reduction Incentive Proposals
CHAPTER 300	GEOMETRIC CROSS SECTION
Topic 303	Curbs, Dikes, and Side Gutters
Index 303.4	Busbulbs
Topic 304	Side Slopes
Index 304.1	Side Slopes – Erosion Control
304.1	Side Slopes – Structural Integrity
309.2	Vertical Clearance on National Highway System
309.2	Vertical Clearance Above Railroad Facilities
309.5	Horizontal and Vertical Clearances at Railroad Structures
CHAPTER 500	TRAFFIC INTERCHANGES
Topic 502	Interchange Types
Index 502.2	Other Types of Interchanges
Topic 503	Interchange Procedure
Index 503.2	Interchange Geometrics

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Table 82.1C

Decision Requiring Other Approvals (Cont.)

Topic 504	Interchange Design Standards
Index 504.3	HOV Preferential Lane
504.3	Modification to Existing HOV Preferential Lanes
504.3	Enforcement Areas and Maintenance Pullouts – Required Enforcement Area
504.3	Enforcement Areas and Maintenance Pullouts – Removal
504.3	Enforcement Areas and Maintenance Pullouts - Length
504.6	Mainline Lane Reduction
CHAPTER 600	PAVEMENT ENGINEERING
Topic 604	Roles, Resources, and Proprietary Items
Index 604.3	Pavement Recommendations
604.4	Other Resources
Topic 606	Research and Special Designs
Index 606.1	Research and Experimentation
CHAPTER 610	PAVEMENT ENGINEERING CONSIDERATIONS
Topic 614	Soil Characteristics
Index 614.6	Other Considerations
CHAPTER 620	RIGID PAVEMENT
Topic 626	Other Considerations
Index 626.2	Shoulder – Widened Slab
CHAPTER 700	MISCELLANEOUS STANDARDS
Topic 701	Fences
Index 701.2	Locked Gates - Maintenance Force Use*
701.2	Locked Gates – Utility Companies, Non-Utility Entities, or Public Agencies*

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

Table 82.1C

Decision Requiring Other Approvals (Cont.)

Topic 706	Roadside Management and Vegetation Control
Index 706.2	Vegetation Control
CHAPTER 800	HIGHWAY DRAINAGE DESIGN
Topic 805	Preliminary Plans
Index 805.1	Requires FHWA Approval
805.2	Bridge Preliminary Report
805.4	Unusual Hydraulic Structures
805.5	Levees and Dams Formed by Highway Fills
805.6	Geotechnical
Topic 808	Selected Computer Programs
Index 808.1	Table 808.1
CHAPTER 820	CROSS DRAINAGE
Topic 829	Other Considerations
Index 829.9	Dams
CHAPTER 830	TRANSPORTATION FACILITY DRAINAGE
Topic 837	Inlet Design
Index 837.2	Inlet Types
CHAPTER 850	PHYSICAL STANDARDS
Topic 853	Pipe Liners and Linings for Culvert Rehabilitation
Index 853.4	Alternative Pipe Liner Materials
CHAPTER 870	CHANNEL AND SHORE PROTECTION – EROSION CONTROL
Topic 872	Planning and Location Studies
Index 872.3	Site Consideration
Topic 873	Design Concepts
Index 873.1	Introduction
873.3	Armor Protection

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

Table 82.1C

Decision Requiring Other Approvals (Cont.)

CHAPTER 900	LANDSCAPE ARCHITECTURE - ROADSIDES
Topic 904	Planting Design
Index 904.1	Planting Design General
Index 904.3	Plant Selection
Topic 905	Irrigation Design
Index 905.1	Irrigation Design General
Index 905.4	Irrigation System Equipment
CHAPTER 910	LANDSCAPE ARCHITECTURE – ROADSIDE SITES
Topic 912	Roadside Sites Design
Index 912.1	Roadside Sites Layout
Index 912.3	Site Furnishings
Topic 913	Safety Roadside Rest Areas
Index 913.4	Safety Roadside Rest Area Buildings and Structures
Index 913.5	Safety Roadside Rest Area Utilities and Facilities
Topic 914	Vista Points
Index 914.3	Vista Point Amenities
Topic 915	Park & Ride Facilities
Index 915.1	Park & Ride Facilities General
CHAPTER 1000	BICYCLE TRANSPORTATION DESIGN
Topic 1003	Miscellaneous Criteria
Index 1003.5	Bicycle Path at Railroad Crossings
CHAPTER 1100	HIGHWAY TRAFFIC NOISE ABATEMENT
Topic 1101	General Requirements
Index 1101.2	Objective – Extraordinary Abatement

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

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

Topic 103 – Design Designation

103.1 Relation to Design

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

ADT (2015) = 9800 D = 60 %

ADT (2035) = 20 000 T = 12 %

DHV = 3000 V = 70 mph

ESAL = 4 500 000 TI_{20} = 11.0

CLIMATE REGION = Desert

The notation above is explained as follows:

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

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

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

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

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

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

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

TI_{20} -- Traffic Index used for pavement engineering. The number in the subscript is the pavement design life used for pavement design. See Index 613.3(5).

V -- Design speed in miles per hour.

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

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

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

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

The design designation should be stated in project initiation documents and project reports and should appear on the typical cross section for all new, reconstructed, or rehabilitation (including Capital Preventative Maintenance) highway construction projects.

103.2 Design Period

Geometric design of new facilities and reconstruction projects should typically be based on estimated traffic 20 years after completion of construction. For new facilities and reconstruction projects on the Interstate System a minimum 20-year design period is required. With justification, for projects other than on the Interstate System, design periods less than 20 years may be approved by the District Director with concurrence by the Project Delivery Coordinator.

For roundabout design period guidance, see Index 405.10.

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

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

Topic 104 – Control of Access

104.1 General Policy

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

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

104.2 Access Openings

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

(3) Procedures.

(a) The engineer will consider pedestrian accessibility needs in the project initiation documents for all projects where applicable.

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

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

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

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

105.5 Guidelines for the Location and Design of Curb Ramps

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

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

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

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

When curb ramps are constructed or reconstructed, one curb ramp should be provided for each pedestrian street crossing. A blended transition or wide curb ramp with street transitions for the direction of both pedestrian crossings serves the purpose of two curb ramps. For example, at intersection corners where two pedestrian street crossings are located, two curb ramps should be constructed; if only one pedestrian street crossing is located at a corner, one curb ramp may be constructed. See Index 105.6 for further information. The usage of the one-ramp design should be restricted to those locations where the volume of pedestrians and vehicles making right turns is low. This will reduce the potential frequency of conflicts between turning vehicles and persons with disabilities entering the common crosswalk area to cross either street.

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

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

105.6 Pedestrian Crossings

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

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

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

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

- Section 275 - For the definition of crosswalk, see Index 62.4(5). Section 275 describes marked and unmarked crosswalks.
- Section 360 - A highway is a way or place of whatever nature, publicly maintained and open to the use of the public for purposes of vehicular travel. Highway includes street.
- Section 365 - An "intersection" is the area embraced within the prolongations of the lateral curb lines, or, if none, then the lateral boundary lines of the roadways, of two highways which join one another at approximately right angles or the area within which vehicles traveling upon different highways joining at any other angle may come in conflict.
- Section 530 - A "roadway" is that portion of a highway improved, designed, or ordinarily used for vehicular travel.
- Section 555 - A "sidewalk" is that portion of a highway, other than the roadway, set apart by curbs, barriers, markings or other delineation for pedestrian travel.

Topic 106 – Stage Construction and Utilization of Local Roads

106.1 Stage Construction

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

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

114.3 Content

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

(1) *Pavement.* At minimum, the Materials Report must document the material data to be used to engineer the pavement structure, including the following:

- Engineering studies, tests, and cores performed to collect data for the project.
- Deflection studies for existing flexible pavement rehabilitation projects (see Index 635.2(2), and California Test Method 357).
- Special material requirements that should be incorporated such as justifications for using (or not using) particular materials in the pavement structure.
- Pavement strategy/structural recommendations are not included as part of the Materials Report. See Index 604.3 for discussion on preparation of pavement recommendations.

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

(3) *Corrosion.* Corrosion studies are necessary when new culverts, culvert rehabilitation, or culvert extensions are part of the scope of the project. Studies should satisfy the requirements of the "Corrosion Guidelines". Copies of the guidelines can be obtained from the Corrosion Technology Branch in DES Materials Engineering and Testing Services or on the DES Materials Engineering and Testing Services website.

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

114.4 Preliminary Materials Report

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

114.5 Review and Retention of Records

A copy of the Draft Materials Report is to be submitted for review and comment to the District Materials Engineer. The District Materials Engineer reviews the document for the Department to assure that it meets the standards, policies, and other requirements found in Department manuals, and supplemental district guidance (Index 604.1(3)). If it is found that the document meets these standards, the District Materials Engineer accepts the Materials Report. If not, the report is returned with comments to the submitter.

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

Topic 115 – Designing for Bicycle Traffic

115.1 General

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

All city, county, regional and other local agencies responsible for bikeways or roads except those freeway segments where bicycle travel is prohibited shall follow the bikeway design criteria established in this manual and the California MUTCD, as authorized in the Streets and Highways Code Sections 890.6 and 891(a). However, a local agency may utilize alternative design criteria as prescribed in the Streets and Highways Code Section 891(b). The decision to develop bikeways should be made in consultation and coordination with local agencies responsible for bikeway planning to ensure connectivity and network development.

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

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

accommodation of pedestrians and bicyclists, the width of approach roadbed that will exist at the time the bridge is constructed, traffic volumes, needs of the local agencies, controls in the form of existing facilities, and the practical challenges of falsework construction.

The normal width of traffic openings and required falsework spans are shown in Table 204.8.

The normal spans shown in Table 204.8 are for anchored temporary barrier. When temporary barrier is not anchored, add 8 feet minimum to normal span to include barrier deflection.

The minimum vertical falsework clearance over freeways and nonfreeways shall be 15 feet. The following items should be considered:

- Mix, volume, and speed of traffic.
- Effect of increased vertical clearance on the grade of adjacent sections.
- Closing local streets to all traffic or trucks only during construction.
- Detours.
- Carrying local traffic through construction on subgrade.
- Temporary or permanent lowering of the existing facility.
- Cost of higher clearance versus cost of traffic control.
- Desires of local agency.

Worker safety should be considered when determining vertical falsework clearance. Requests for approval of temporary vertical clearances less than 15 feet should discuss the impact on worker safety.

Temporary horizontal clearances less than shown in Table 204.8 or temporary vertical clearances less than 15 feet should be noted in the PS&E Transmittal Report.

To establish the grade of a structure to be constructed with a falsework opening, allowance must be made for the depth of the falsework. The minimum depths required for various widths of traffic opening are shown in Table 204.8.

Where vertical clearances, either temporary or permanent are critical, the District and the DES – Structure Design should work closely during the early design stage when the preliminary grades, structure depths, and falsework depths can be adjusted without incurring major design changes.

Where the vertical falsework clearance is less than 15 feet, advance warning devices are to be specified or shown on the plans. Such devices may consist of flashing lights, overhead signs, over-height detectors, or a combination of these or other devices.

Warning signs on the cross road or in advance of the previous off-ramp may be required for overheight permit loads. Check with the Regional Permit Manager.

After establishing the opening requirements, a field review of the bridge site should be made by the District designer to ensure that existing facilities (drainage, other bridges, or roadways) will not conflict with the falsework.

The placement and removal of falsework requires special consideration. During these operations, traffic should either be stopped for short intervals or diverted away from the

Table 204.8

Falsework Span and Depth Requirements

Facility to be Spanned	Minimum Normal Width of Traffic Opening (2)(3)(4)	Resulting Falsework	Depth of Superstructure(5) Up to 6 feet	Up to 8 feet	Up to 10 feet	Up to 12 feet
		Normal Span(1)	Minimum Falsework Depth			
Freeway & Non Freeway	20'	31'	1'-10"	2'-0"	2'-0"	2'-4"
	25'	36'	1'-11"	2'-4"	2'-4"	2'-10"
	32'	43'	2'-5"	2'-11"	2'-11"	3'-2"
	37'	48'	3'-0"	3'-1"	3'-3"	3'-3"
	40'	51'	3'-1"	3'-2"	3'-3"	3'-3"
	49'	60'	3'-3"	3'-3"	3'-3"	3'-4"
	52'	63'	3'-4"	3'-3"	3'-3"	3'-5"
	61'	72'	3'-5"	3'-8"	3'-8"	3'-9"
	64'	75'	3'-5"	3'-8"	3'-8"	3'-9"
	73'	84'	3'-8"	3'-9"	3'-9"	3'-10"

NOTES:

(1) Includes 11' for two temporary barriers and 3.5' to center line of falsework post (12 inch post assumed). This is a minimum clearance for barriers with the maximum number of required anchors. Additional span distance may be required depending on temporary barrier system used and its configuration. See RSS 12-3.20 for additional information.

(2) Approach roadway width measured normal to lanes. Use next highest width if the approach roadway width is not shown in the table.

(3) Dependent upon the width of approach roadbed available at the time of bridge construction.

(4) Clear vehicular opening between temporary railings.

(5) See Index 204.8 for preliminary depth to span ratios. For more detailed information, contact the Division of Engineering Services, Structure Design and refer to the Bridge Design Aids.

(6) Distances rounded to nearest inch.

water is carried away from the structure approach slab at a location where it will not cause erosion. The PE is responsible for the engineering of the outlet for the structure approach slab drainage. Storm Water Best Management Practices should be considered.

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

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

(4) Transition Details with Pavement Overlays. Modification to structure approach slab thicknesses are advantageous when structure approach slabs will be replaced in conjunction with a pavement overlay strategy to promote a smooth transition between structure and pavement. Figure 209.4B, which is applicable to full-width slab replacement, illustrates a method of transitioning from an asphalt overlay thickness to a structure approach slab by tapering the thickness of the structure approach slab. Care should be taken in areas with flat grades to avoid creating a ponding condition at the structure abutment.

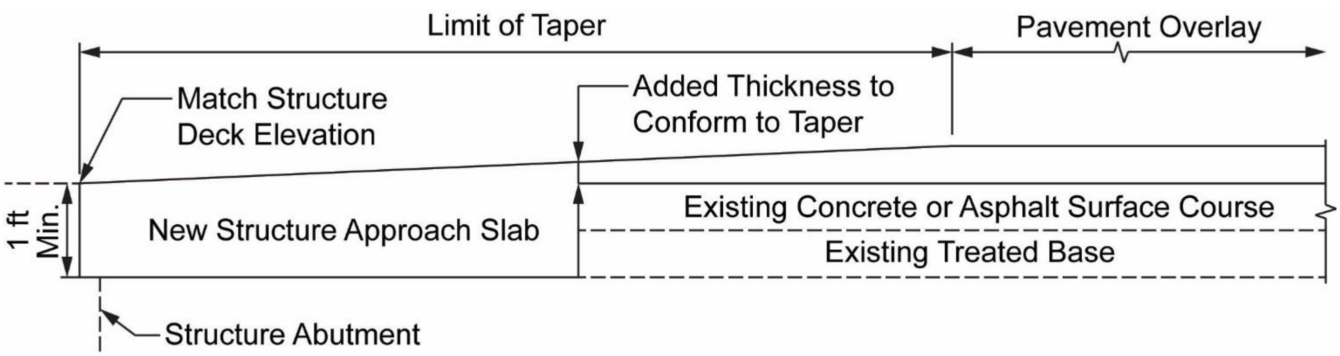
(5) Traffic Handling. Traffic handling considerations typically preclude full-width construction procedures. Structure approach rehabilitation is therefore usually done under traffic control conditions, which require partial-width construction.

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

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

Figure 209.4B

New Structure Approach Pavement Transition Details



Topic 210 – Reinforced Earth Slopes and Earth Retaining Systems

210.1 Introduction

Constructing roadways on new alignments, widening roadways on an existing alignment, or repairing earth slopes damaged by landslides are situations that may require the use of reinforced earth slopes or earth retaining systems. Using cut and embankment slopes that are configured at slope ratios that are stable without using reinforcement is usually preferred; however, topography, environmental concerns, and right of way (R/W) limitations may require the need for reinforced earth slopes or an earth retaining system.

The need for reinforced earth slopes or an earth retaining system should be identified as early in the project development process as possible, preferably during the Project Initiation Document (PID) phase.

210.2 Construction Methods and Types

(1) Construction Methods

Both reinforced earth slopes and earth retaining systems can be classified by the method in which they are constructed, either top-down or bottom-up.

- "Top-down" construction – This method of construction begins at the top of the reinforced slope or earth retaining system and proceeds in lifts to the bottom of the reinforced slope or earth retaining system.

  If required, reinforcement is inserted into the in situ material during excavation.

- "Bottom-up" construction – This method of construction begins at the bottom of the reinforced slope or earth retaining system, where a footing/leveling pad is constructed, construction then proceeds towards the top of the reinforced slope or earth retaining system. If required, reinforcement is placed behind the face of the reinforced slope or earth retaining system. It should be noted that if a "Retaining Wall" earth retaining system is to be used in a cut situation, a temporary back cut or shoring system is required behind the wall.

The District Project Engineer (PE) should conduct an initial site visit and assessment to determine all potential construction limitations. The preferred construction method is top-down due to the reduced shoring, excavation and backfilling. However, this method is not always available or appropriate based on the physical and geotechnical site conditions. The site should also be examined for R/W or utility constraints that would restrict the type of excavation or limit the use of some equipment. In addition, the accessibility to the site for construction and contractor staging areas should be considered.

Table 210.2 summarizes the various reinforced earth slopes and earth retaining systems that are currently available for use, along with the method in which they are constructed.

Table 210.2

Types of Reinforced Earth Slopes and Earth Retaining Systems(1)

EARTH RETAINING SYSTEM	Construction Method(2)	PS&E By	Typical Facing Material	Recommended Maximum Vertical Height, ft	Ability to Tolerate Differential Settlement(3)
Reinforced Earth Slopes					
Reinforced Embankments	BU	District PE	Vegetation/Soil	160	E
Rock/Soil Anchors	TD	District PE	Soil/Rock	130	E
State Designed Earth Retaining Systems with Standard Plans					
Concrete Cantilever Wall, Type 1 & 1A	BU	District PE	Concrete	36, 12, 22(4)	P
Concrete L-Type Cantilever Wall, Type 5	BU	District PE	Concrete	12(4)	P
Concrete Masonry Wall, Type 6	BU	District PE	Masonry	6(4)	P
State Designed Earth Retaining Systems Which Require Special Designs					
Standard Plan Walls with modified wall geometry, foundations or loading conditions	BU	Structure PE	Concrete, Steel, Timber	50	P-F
Non-Gravity Cantilevered Walls					
Sheet Pile Wall	TD	Structure PE	Steel	20	F
Soldier Pile Wall with Lagging	TD/BU	Structure PE	Concrete, Steel, Timber	20	F-G
Tangent Soldier Pile Wall	TD/BU	Structure PE	Concrete	30	F
Secant Soldier Pile Wall	TD	Structure PE	Concrete	30	F
Slurry Diaphragm Wall	TD	Structure PE	Concrete, Shotcrete	80(5)	F
Deep Soil Mixing Wall	TD	Structure PE	Shotcrete	80(5)	F-G
Anchored Wall (Structural or Ground Anchors)	TD	Structure PE	Concrete, Steel, Timber	80(6)	F-G
Gravity Walls					
Concrete Gravity Wall	BU	Structure PE	Concrete	6	P
Rock Gravity Wall	BU	District PE	Rock	13	E
Gabion Basket Wall	BU	District PE	Wire & Rock	26	E
Soil Reinforcement Systems					
Mechanically Stabilized Embankment	BU	Structure PE	Concrete	50	G
Salvaged Material Retaining Wall	BU	District PE	Steel, Timber	16	G
Soil Nail Wall	TD	Structure PE	Concrete, Shotcrete	80	F
Tire Anchored Timber Wall	BU	District PE	Timber	32	G
Proprietary Earth Retaining Systems (Pre-approved)					
The list of Pre-approved systems is available at the website shown in Index 210.2(3)(c).					
Proprietary Earth Retaining Systems (Pending)					
These systems are under review by DES-SD. For more information, see Index 210.2(3)(d).					
Experimental State Designed Earth Retaining Systems					
Geosynthetic Reinforced Walls	BU	Structure PE/ District PE	Concrete Blocks, Steel, Vegetation, Fabric	65	E
Mortarless Concrete Blocks Gravity Walls	BU	District PE	Concrete Blocks	8	P

NOTES:
1. Comparative cost data is available from DES-SD.
2. BU = Bottom Up; TD = Top Down
3. E = Excellent; G = Good; F = Fair; P = Poor
4. Maximum Design Height
5. Anchors may be required
6. With lagging

(2) Reinforced Earth Slopes (PS&E by District PE)

Reinforced earth slopes incorporate metallic or non-metallic reinforcement in construction of embankments and cut slopes with a slope angle flatter than 70 degrees from the horizontal plane. Reinforced earth slopes should be used in conjunction with erosion mitigation measures to minimize future maintenance costs. The slope face is typically erosion protected with the use of systems such as geosynthetics, bio-stabilization, rock slope protection, or reinforced concrete facing.

(3) Earth Retaining Systems

Earth retaining systems can be divided into five major categories depending upon the nature of the design and whether they are designed by the owner (State designed), a Proprietary vendor or a combination thereof. The term “State designed” as referenced herein is utilized to encompass earth retaining systems that are designed by the State or by Local or Private entities on behalf of the State.

No assignment of roles and responsibilities is intended. The five categories are as follows:

(a) State Designed Earth Retaining Systems which utilize Standard Plans (PS&E by District PE).

Standard Plans are available for a variety of earth retaining systems (retaining walls). Loading conditions and foundation requirements are as shown on the Standard Plans. For sites with requirements that are not covered by the Standard Plans, a special design is required. To assure conformance with the specific Standard Plan conditions and requirements, and subsequent completion of the PS&E in a timely fashion, the District PE should request a foundation investigation for each location where a retaining wall is being considered. Retaining walls that utilize Standard Plans are as follows:

- Retaining Wall Types 1 and 1A (Concrete Cantilever). These walls have design heights up to 36 feet and 12 feet respectively, but are most economical below 20 feet. Concrete cantilever walls can accommodate traffic barriers, and drainage facilities efficiently. See Standard Plans for further details.
- Retaining Wall Type 5 (Concrete L-Type Cantilever). This wall has a design height up to 12 feet. Although more costly than cantilever walls, these walls may be required where site restrictions do not allow for a footing projection beyond the face of the wall stem. See Standard Plans for further details.
- Retaining Wall Type 6 (Concrete Masonry Walls). These walls may be used where the design height of the wall does not exceed 6 feet. These walls are generally less costly than all other standard design walls or gravity walls. Where traffic is adjacent to the top of the wall, guardrail should be set back as noted in the Standard Plans. See Standard Plans or further details.

Topic 302 – Highway Shoulder Standards

302.1 Width

The shoulder widths given in Table 302.1 shall be the minimum continuous usable width of paved shoulder on highways. Typically, on-street parking areas in urbanized areas is included in the shoulder.

When present, Class II bikeways are typically part of the shoulder width, see Index 301.2.

See Rumble Strip Guidelines, Traffic Safety Bulletin 20-07 for placement of rumble strips and rumble stripes in and adjacent to shoulders. Consult the District Traffic Safety Engineer during selection of rumble strip and stripe options and refer to the California MUTCD for markings in combination with rumble strip and stripe. Also see Standard Plans for rumble strip and stripe details.

See DIB 79 for 2R, 3R, certain storm damage, protective betterment, operational, and safety projects on two-lane and three-lane conventional highways.

See Index 308.1 for shoulder width requirements on city streets or county roads. See shoulder definition, Index 62.1(9).

See Index 1102.2 for shoulder width requirements next to noise Barriers.

When shoulders are less than standard width, see Index 204.5(4) for bicycle turnout considerations.

302.2 Cross Slopes

(1) *General* - **When a roadway crosses a bridge structure, the shoulders shall be in the same plane as the adjacent traveled way**.

(2) *Left Shoulders* - **In depressed median sections, shoulders to the left of traffic shall be sloped at 2 percent away from the traveled way.**

    In paved median sections, shoulders to the left of traffic shall be designed in the plane of the traveled way. Maintenance paving beyond the edge of shoulder should be treated as appropriate for the site, but consideration needs to be given to the added runoff and the increased water depth on the pavement (see discussion in Index 831.4(5) "Hydroplaning").

(3) *Right Shoulders*- In normal tangent sections, **shoulders to the right of traffic shall be sloped at 2 percent to 5 percent away from the traveled way.**

    The above flexibility in the design of the right shoulder allows the designer the ability to conform to regional needs. Designers shall consider the following during shoulder cross slope design:

Table 302.1

Boldface Standards for Paved Shoulder Widths on Highways

Highway Type	Paved Shoulder Width (ft) Left (8)	Right (8)
Freeways & Expressways		
2 lanes (1)	--	8(6)
4 lanes (1)	5	10
6 or more lanes (1)	10	10
Auxiliary lanes	--	10
Freeway-to-freeway connections		
Single and two-lane connections	5	10
Three-lane connections	10	10
Single-lane ramps	4(2)	8
Multilane ramps	4(2)	8(3)
Multilane undivided	--	10
Collector-Distributor	5	10
Conventional Highways		
Multilane divided		
4-lanes	5	8(7)
6-lanes or more	8	8(7)
Urban areas with posted speeds less than or equal to 45 mph and curbed medians	2(4)	8(7)
Multilane undivided	--	8(7)
2-lane		
RRR	See Index 307.3	
New construction	See Table 307.2	
Slow-moving vehicle lane	--	4(5)
Local Facilities		
Frontage roads	See Index 310.1	
Local facilities crossing State facilities	See Index 308.1	

NOTES:

(1) Total number of lanes in both directions including separate roadways (see Index 305.6). If a lane is added to one side of a 4-lane facility (such as a truck climbing lane) then that side shall have 10 feet left and right shoulders. See Index 62.1.

(2) May be reduced to 2 feet upon concurrence from the Project Delivery Coordinator that a restrictive situation exists. 4 feet preferred in urban areas and/or when ramp is metered. See Index 504.3.

(3) May be reduced to 2 feet or 4 feet (4 feet preferred in urban areas) in the 2-lane section of a non-metered ramp, which transitions from a single lane upon concurrence from the Project Delivery Coordinator that a restrictive situation exists. May be reduced to 2 feet in ramp sections having 3 or more lanes. See Index 504.3.

(4) For posted speeds less than or equal to 35 mph, shoulder may be omitted (see Index 303.5(5)) except where drainage flows toward the curbed median.

(5) On right side of climbing or passing lane section only. See Index 301.2(1) for minimum width if bike lanes are present.

(6) 10-foot shoulders preferred.

(7) Where on-street parking is allowed, 10 feet shoulder width is preferred. Where bus stops are present, 10 feet shoulder width is preferred for the length of the bus stop. If a Class II bikeway is present, minimum shoulder width shall be 8 feet where on street parking is provided plus the minimum required width for the bike lane.

(8) Shoulders adjacent to abutment walls, retaining walls in cut locations, and noise barriers shall be not less than 10 feet wide. See Index 303.4 for minimum shoulder adjacent to bulbouts. See Index 309.1(4) for minimum shoulder width adjacent to high speed rail facilities.

- In most areas a 5 percent right shoulder cross slope is desired to most expeditiously remove water from the pavement and to allow gutters to carry a maximum water volume between drainage inlets. The shoulders must have adequate drainage interception to control the "water spread" as discussed in Table 831.3 and Index 831.4. Conveyance of water from the total area transferring drainage and rainwater across each lane and the quantity of intercepting drainage shall also be a consideration in the selection of shoulder cross slope. Hydroplaning is discussed in Index 831.4 (5).

  In locations with snow removal operations it is desirable for right shoulders to slope away from traffic in the same plane as the traveled way. This design permits the snow plowing crew to remove snow from the lanes and the shoulders with the least number of passes.
- For 2-lane roads with 4-foot shoulders, see Index 307.2.
- If shoulders are Portland cement concrete and the District plans to convert shoulders into through lanes within the 20 years following construction, then shoulders are to be built in the plane of the traveled way and to lane standards for width and structural section. (See Index 603.2).
- Deciding to construct pedestrian facilities and elements, where none exist, is an important consideration. Shoulders are not required to be designed as accessible pedestrian routes although it is legal for a pedestrian to traverse along a highway. In urban, rural main street areas, or near schools and bus stops with pedestrians present, pedestrian facilities should be constructed. In rural areas where few or no pedestrians exist, it would not be reasonable or cost effective to construct pedestrian facilities. This determination should involve the local agency and must be consistent with the design guidance provided in Topic 105 and in Design Information Bulletin 82, "Pedestrian Accessibility Guidelines for Highway Projects" for people with disabilities.

Shoulder slopes for superelevated curves are discussed in Index 202.2.

See Index 307.2 for shoulder slopes on 2-lane roads with 4-foot shoulders.

302.3 Tapered Edge

The tapered edge is a sloped edge that is placed at the edge of the paved roadbed to provide a smooth reentry for vehicles that leave the roadway. Its design is based on research performed by the FHWA.

The tapered edge should be placed on all pavement edges either during new construction or on overlay projects irrespective of pavement types and is most useful:

- On undivided roadways.
- On roadways with unpaved shoulders.
- On roadways with Class II Bikeways.

The tapered edge is not to be placed on roadways:

- Next to curbs, dikes, guardrails, barriers, walls, and landscape paving.
- Where there is not enough room to place the tapered edge without reducing the existing lane width.

- Within 3 feet of driveways or intersections.
- Where pavement overlay thickness is less than 0.15 foot.

Tapered edge is optional when the distance between consecutive minor roads or driveways is less than 30 feet. See the Standard Plans for design and construction details regarding tapered edge.

Topic 303 – Curbs, Dikes, and Side Gutters

303.1 General Policy

Curb (including curb with gutter pan), dike, and side gutter all serve specific purposes in the design of the roadway cross section. Curb is primarily used for channelization, access control, separation between pedestrians and vehicles, and to enhance delineation. Dike is specifically intended for drainage and erosion control where stormwater runoff cannot be cost effectively conveyed beyond the pavement by other means. Curb with gutter pan serves the purpose of both curb and dike. Side gutters are intended to prevent runoff from a cut slope on the high side of a superelevated roadway from running across the pavement and is discussed further in Index 834.3.

Aside from their positive aspects in performing certain functions, curbs and dikes can have undesirable effects. In general, curbs and dikes should present the least potential obstruction, yet perform their intended function. As operating speeds increase, lower curb and dike height is desirable. Curbs and dikes are not considered traffic barriers.

On urban conventional highways where right of way is costly and/or difficult to acquire, it is appropriate to consider the use of a “closed” highway cross section with curb, or curb with gutter pan. There are also some situations where curb is appropriate in freeway settings. The following criteria describe typical situations where curb or curb with gutter pan may be appropriate:

(a) Where needed for channelization, delineation, or other means of improving traffic flow and safety.

(b) At ramp connections with local streets for the delineation of pedestrian walkways and continuity of construction at a local facility.

(c) As a replacement of existing curb with gutter pan and sidewalk.

(d) On frontage roads on the side adjacent to the freeway to deter vehicular damage to the freeway fence.

(e) When appropriate to conform to local arterial street standards.

(f) Where it may be necessary to solve or mitigate operational deficiencies through control or restriction of access of traffic movements to abutting properties or traveled ways.

(g) In freeway entrance ramp gore areas (at the inlet nose) when the gore cross slope exceeds standards.

(h) At separation islands between a freeway and a collector-distributor to provide a positive separation between mainline traffic and collector-distributor traffic.

(i) Where sidewalk is appropriate.

Topic 305 – Median Standards

305.1 Width

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

a. Public Transit (rail and bus).

b. Traffic needs more than 20 years after completion of construction.

Median width as presented in Case (1) below applies to new construction, projects to increase mainline capacity and to reconstruction projects. Any recommendation to provide additional median width should be identified and documented as early as possible and must be justified in a project initiation document and/or project report. Attention should be given to such items as initial costs, future costs for outside widening, the likelihood of future needs for added mixed flow or High-Occupancy Vehicle (HOV) lanes, traffic interruption, future mass transit needs and right of way considerations. (For instance, increasing median width may add little to the cost of a project where an entire city block must be acquired in any event.)

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

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

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

(1) Freeways and Expressways.

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

minimum median width for multilane freeways and expressways in urban areas should be 36 feet.

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

(2) *Conventional Highways.* Appropriate median widths for non-controlled access highways vary widely with the type of facility being designed. In Urban and Rural Main Street areas, the minimum median width for multilane conventional highways should be 12 feet. However, this width would not provide room for left-turn lanes at intersections with raised curb medians, nor left-turn lanes in striped medians with room for pedestrian refuge areas. Posted speed and left shoulder width can also affect median width. See Table 302.1.

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

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

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

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

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

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

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

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

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

See Index 613.4(2)(b) for paved median pavement structure requirements.

305.2 Median Cross Slopes

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

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

305.3 Median Barriers

See Traffic Safety Systems Guidance.

305.4 Median Curbs

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

305.5 Paved Medians

(1) Freeways.

(a) 6 or More Lanes--Medians 30 feet wide or less should be paved.

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

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

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

For additional information on median cross slopes see Index 305.2.

305.6 Separate Roadways

(1) *General Policy.* Separate grade lines are not considered appropriate for medians less than 65 feet wide (see Index 204.7).

(2) *Median Design.* The cross sections shown in Figure 305.6 include a clear recovery zone that provides maneuvering room for out-of-control users. See Index 309.1(2).

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

Topic 306 – Right of Way

306.1 General Standards

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

306.2 Right of Way Through the Public Domain

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

Topic 307 – Cross Sections for State Highways

307.1 Cross Section Selection

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

When it becomes necessary to widen an existing cross section, e.g., add or widen the paved shoulder or lane, refer to Index 653.2 and Index 662.3 to ensure proper drainage of both the existing and widening structural sections. See also Chapter 680, Pavement Design for Widening Projects.

CHAPTER 400 – INTERSECTIONS AT GRADE

Intersections are planned points of conflict where two or more roadways join or cross. At-grade intersections are among the most complicated elements on the highway system, and control the efficiency, capacity, and safety for motorized and non-motorized users of the facility. The type and operation of an intersection is important to the adjacent property owners, motorists, bicyclists, pedestrians, transit operators, the trucking industry, and the local community.

There are two basic types of at grade intersections: crossing and circular. It is not recommended that intersections have more than four legs. Occasionally, local development and land uses create the need for a more complex intersection design. Such intersections may require a specialized intersection design to handle the specify traffic demands at that location. In addition to the guidance in this manual, see Traffic Operations Policy Directive (TOPD) Number 13-02: Intersection Control Evaluation (ICE) for direction and procedures on the evaluation, comparison and selection of the intersection types and control strategies identified in Index 401.5. Also refer to the publication Complete Intersections: A Guide to Reconstructing Intersections and Interchanges for Bicyclists and Pedestrians for further information.

Topic 401 – Factors Affecting Design

Index 401.1 – General

At-grade intersections must handle a variety of conflicts among users, which includes truck, transit, pedestrians, and bicycles. These recurring conflicts play a major role in the preparation of design standards and guidelines. Arriving, departing, merging, turning, and crossing paths of moving pedestrians, bicycles, truck, and vehicular traffic have to be accommodated within a relatively small area. The objective of designing an intersection is to effectively balance the convenience, ease, and comfort of the users, as well as the human factors, with moving traffic (automobiles, trucks, motorcycles, transit vehicles, bicycles, pedestrians, etc.).The safety and mobility needs of motorist, bicyclist and pedestrians as well as their movement patterns in intersections must be analyzed early in the planning phase and then followed through appropriately during the design phase of all intersections on the State highway. It is Departmental policy to develop integrated multimodal projects in balance with community goals, plans, and values.

The Complete Intersections: A Guide to Reconstructing Intersections and Interchanges for Bicyclists and Pedestrians contains a primer on the factors to consider when designing intersections. It is published by the California Division of Traffic Operations.

401.2 Human Factors

(1) *The Driver.* An appreciation of driver performance is essential to proper highway design and operation. The suitability of a design rests as much on how safely and efficiently drivers are able to use the highway as on any other criterion.

Motorist's perception and reaction time set the standards for sight distance and length of transitions. The driver's ability to understand and interpret the movements and crossing times of the other vehicle drivers, bicyclists, and pedestrians using the intersection is equally important when making decisions and their associated reactions. The designer needs to keep in mind the user's limitations and therefore design intersections so that they meet user expectation.

(2) *The Bicyclist.* Bicyclist experience, skills and physical capabilities are factors in intersection design. Intersections are to be designed to help bicyclists understand how to traverse the intersection. Chapter 1000 provides intersection guidance for Class I and Class III bikeways that intersect the State highway system. The guidance in this chapter specifically relates to bicyclists that operate within intersections on the State highway system.

(3) *The Pedestrian.* Understanding how pedestrians will use an intersection is critical because pedestrian volumes, their age ranges, physical ability, etc. all factor in to their startup time and the time it takes them to cross an intersection and thus, dictates how to design the intersection to avoid potential conflicts with bicyclists and motor vehicles. The guidance in this chapter specifically relates to pedestrian travel within intersections on the State highway system. See Topic 105, Pedestrian Facilities, Design Information Bulletin 82 - "Pedestrian Accessibility Guidelines for Highway Projects," the AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities, and the California Manual on Uniform Traffic Control Devices (California MUTCD) for additional guidance.

401.3 Traffic Considerations

Good intersection design clearly indicates to bicyclists and motorists how to traverse the intersection (see Figure 403.6A). Designs that encourage merging traffic to yield to through bicycle and motor vehicle traffic are desirable.

The size, maneuverability, and other characteristics of bicycles and motorized vehicles (automobiles, trucks, transit vehicles, farm equipment, etc.) are all factors that influence the design of an intersection. The differences in operating characteristics between bicycles and motor vehicles should be considered early in design.

Table 401.3 compares vehicle characteristics to intersection design elements.

A design vehicle is a convenient means of representing a particular segment of the vehicle population. See Topic 404 for a further discussion of the uses of design vehicles.

Transit vehicles and how their stops interrelate with an intersection, pedestrian desired walking patterns and potential transfers to other transit facilities are another critical factor to understand when designing an intersection. Transit stops and their placement needs to take into account the required maintenance operations that will be needed and usually supplied by the Transit Operator.

401.4 The Physical Environment

In highly developed urban areas, where right of way is usually limited, the volume of vehicular traffic, pedestrians, and bicyclists may be large, street parking exists, and transit stops (for both buses and light rail) are available. All interact in a variety of movements that contribute to and add to the complexity of a State highway and can result in busy intersections.

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

Rural areas where farming occurs may require special attention for specialized farm equipment. In addition, rural cities or town centers (rural main streets) also require special attention.

Rural intersections in farm areas with low traffic volumes may have special visibility problems or require shadowing of left-turn vehicles from high speed approach traffic.

Table 401.3

Vehicle Characteristics	Intersection Design Element Affected
Length	Length of storage lane
Width	Lane width
Height	Clearance to overhead signs and signals
Wheel base	Corner radius and width of turning lanes
Acceleration	Tapers and length of acceleration lane
Deceleration	Tapers and length of deceleration lane

There are many factors to be considered in the design of intersections, with the goal to achieve a functional, safe and efficient intersection for all users of the facility. The location and level of use by various modes will have an impact on intersection design, and therefore should be considered early in the design process. In addition to current levels of use, it is important to consider future travel patterns for vehicles, including trucks; pedestrian and bicycle demand and the future expansion of transit.

401.5 Intersection Type

Intersection types are characterized by their basic geometric configuration, and the form of intersection traffic control that is employed:

(1) Geometric Configurations

(a) Crossing-Type Intersections - "Tee" and 4-legged intersections

(b) Circular Intersections –roundabouts, traffic circles, rotaries; however, only roundabouts are acceptable for State highways.

(c) Alternative Intersection Designs – various effective geometric alternatives to traditional designs that can reduce crashes and their severity, improve operations, reduce congestion and delay typically by reducing or altering the number of conflict points; these alternatives include geometric design features such as intersections with displaced left-turns or variations on U-turns. For more information on alternative intersection designs see FHWA's Intersection Safety webpage: https://safety.fhwa.dot.gov/intersection/.

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

(a) Two-Way Stop Controlled - for minor road traffic

(b) All-Way Stop Control

(c) Signal Control

(d) Yield Control (Roundabout)

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

401.6 Transit

Transit use may range from periodic buses, handled as part of the normal mix of vehicular traffic, to Bus Rapid Transit (BRT) or light rail facilities which can have a large impact on other users of the intersection. Consideration of these modes should be part of the early planning and design of intersections.

Topic 402 – Operational Features Affecting Design

402.1 Capacity

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

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

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

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

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

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

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

403.10 Installation of Traffic Control Devices

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

403.11 Summary

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

403.12 Other Considerations

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

Topic 404 – Design Vehicles

404.1 General

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

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

404.2 Design Considerations

It may not be necessary to provide for design vehicle turning movements at all intersections along the State route if the design vehicle's route is restricted or it is not expected to use the cross street frequently. Discuss with Traffic Operations and the local agency before a turning movement is not provided. The goal is to minimize possible conflicts between vehicles, bicycles, pedestrians, and other users of the roadway, while providing the minimum curb radii appropriate for the given situation.

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

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

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

(1) *Traveled way.*

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

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

(2) *Shoulders.* Both tracking width and swept width lines may encroach onto paved shoulders to accommodate turning. For design projects where the tracking width lines are shown to encroach onto paved shoulders, the shoulder pavement structure should be engineered to sustain the weight of the design vehicle. See Index 613 for general traffic loading considerations and Index 626 for tied rigid shoulder guidance. At corners where no sidewalks are provided and pedestrians are using the shoulder, a paved refuge area may be provided outside the swept width of turning vehicle.

(3) *Curbs and Gutters.* Tires may not mount curbs. If curb and gutter are present and any portion of the gutter pan is likewise encroached, the gutter pan must be engineered to match the adjacent shoulder pavement structure. See Index 613.4(2)(h) for gutter pan design guidance.

(4) *Edge of Pavement*. To accommodate a turn, the swept width lines may cross the edge of pavement provided there are no obstructions. The tracking width lines must remain on the pavement structure, including the shoulder, provided that the shoulder is designed to support vehicular traffic. If truck volumes are high, consideration of a wider shoulder is encouraged in order to preserve the pavement edge.

(5) *Bicycle Lanes.* Where bicycle lanes are considered, the design guidance noted above applies. Vehicles are permitted to cross a bicycle lane to initiate or complete a turning movement or for emergency parking on the shoulder. See the California MUTCD for Class II bike lane markings.

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

(6) *Sidewalks.* Tracking width and swept width lines must not encroach onto sidewalks or pedestrian refuge areas, without exception.

(7) *Obstacles.* Swept width lines may not encroach upon obstacles including, but not limited to, curbs, islands, sign structures, traffic delineators/channelizers, traffic signals, lighting poles, guardrails, trees, cut slopes, and rock outcrops.

(8) *Appurtenances.* Swept width lines do not include side mirrors or other appurtenances allowed by the California Vehicle Code, thus, accommodation to non-motorized users of the facility and appurtenances should be considered.

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

Note that the STAA Design Vehicle has a template with a 56-foot (minimum) and a 67-foot (longer) radius and the California Legal Design Vehicle has a template with 50-foot (minimum) and 60-foot (longer) radii. These templates are shown in Figures 404.5A through 404.5D. The longer radius templates are more conservative. The longer radius templates develop less swept width and leave a margin of error for the truck driver. The longer radius templates should be used for conditions where the vehicle may not be required to stop before entering the intersection.

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

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

The Caltrans Truck Network Maps should be consulted for the appropriate design vehicle. See the link at https://dot.ca.gov/programs/traffic-operations/legal-truck-access/truck-network-map.

404.3 Design Tools

District Truck Managers should be consulted early in the project to ensure compliance with the design vehicle guidance contained in Topic 404. Consult local agencies to verify the location of local truck routes. Essentially, two options are available – templates or computer software.

- The turning templates in Figures 404.5A through G are a design aid for determining the swept width and/or tracking width of large vehicles as they maneuver through a turn. The templates can be used as overlays to evaluate the adequacy of the geometric layout of a curve or intersection when reproduced on clear film and scaled to match the highway drawings. These templates assume a vehicle speed of less than 10 miles per hour.
- Computer software such as AutoTURN or AutoTrak can draw the swept width and/or tracking width along any design curve within a CADD drawing program such as MicroStation or AutoCAD. Dimensions taken from the vehicle diagrams in Figures 404.5A through G may be inputted into the computer program by creating a custom vehicle if the vehicle is not already included in the software library. The software can also create a vehicle turn template that conforms to any degree curve desired.

404.4 Design Vehicles and Related Definitions

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

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

(b) STAA Design Vehicle. The STAA design vehicle is a truck tractor-semitrailer combination with a 48-foot semitrailer, a 43-foot kingpin-to-rear-axle (KPRA) distance, an 8.5-foot body and axle width, and a 23-foot truck tractor wheelbase. Note, a truck tractor is a non-load-carrying vehicle. There is also a STAA double (truck tractor-semitrailer-trailer); however, the double is not used as the design vehicle due to its shorter turning radius. The STAA Design Vehicle is shown in Figures 404.5A and B.

<u>The STAA Design Vehicle in Figures 404.5A or B should be used on the National Network, Terminal Access, California Legal, and Advisory routes.</u>

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

(2) California Legal.

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

car driver's eye height should be applied to all minor roads. In addition, a truck driver's eye height of 7.6 feet should be applied to the minor road where applicable. Additionally, if the major road has a median barrier, a 2-foot object height should be used to determine the median barrier set back. A median that is wide enough to accommodate a stopped vehicle should also provide a clear sight triangle.

The minimum corner sight distance (feet) should be determined by the equation: $1.47V_mT_g$, where V_m is the design speed (mph) of the major road and T_g is the time gap (seconds) for the minor road vehicle to enter the major road. The values given in Table 405.1A should be used to determine T_g based on the design vehicle, the type of maneuver, and whether the stopped vehicle's rear wheels are on an upgrade exceeding 3 percent. The distance from the edge of traveled way to the rear wheels at the minor road stop location should be assumed as: 20 feet for a passenger car, 30 feet for a single-unit truck, and 72 feet for a combination truck.

(b) Public Road Intersections (Refer to Topic 205 and Index 405.7); corner sight distance applies, see Table 405.1A.

At signalized intersections the corner sight distances should also be applied whenever possible. Even though traffic flows are designed to move at separate times, unanticipated conflicts can occur due to violation of signal, right turns on red, malfunction of the signal, or use of flashing red/yellow mode.

The minimum value for corner sight distance at signalized intersections should be equal to the stopping sight distance as given in Table 201.1, measured as previously described. This includes an urban driveway that forms a leg of the signalized intersection.

(c) Private Road Intersections (Refer to Index 205.2) and Rural Driveways (Refer to Index 205.4); corner sight distance applies, see Table 405.1A. If signalized, the minimum corner sight distance should 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 unless signalized. See Index 405.1(2)(b) underlined standard. If parking is allowed on the major road, parking should be prohibited on both sides of the driveway per the California MUTCD, 3B.19.

(3) Decision Sight Distance. At intersections where the State route turns or crosses another State route, the decision sight distance values given in Table 201.7 should be used. In computing and measuring decision sight distance, the 3.5-foot eye height and the 0.5-foot object height should be used, the object being located on the side of the intersection nearest the approaching driver.

The application of the various sight distance requirements for the different types of intersections is summarized in Table 405.1B

Table 405.1B

Application of Sight Distance Requirements

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

NOTES:

(1) Per Index 405.1(2)(c), if signalized, the minimum corner sight distance shall be equal to the stopping sight distance as given in Table 201.1. See Index 405.1(2)(a) for setback requirements.

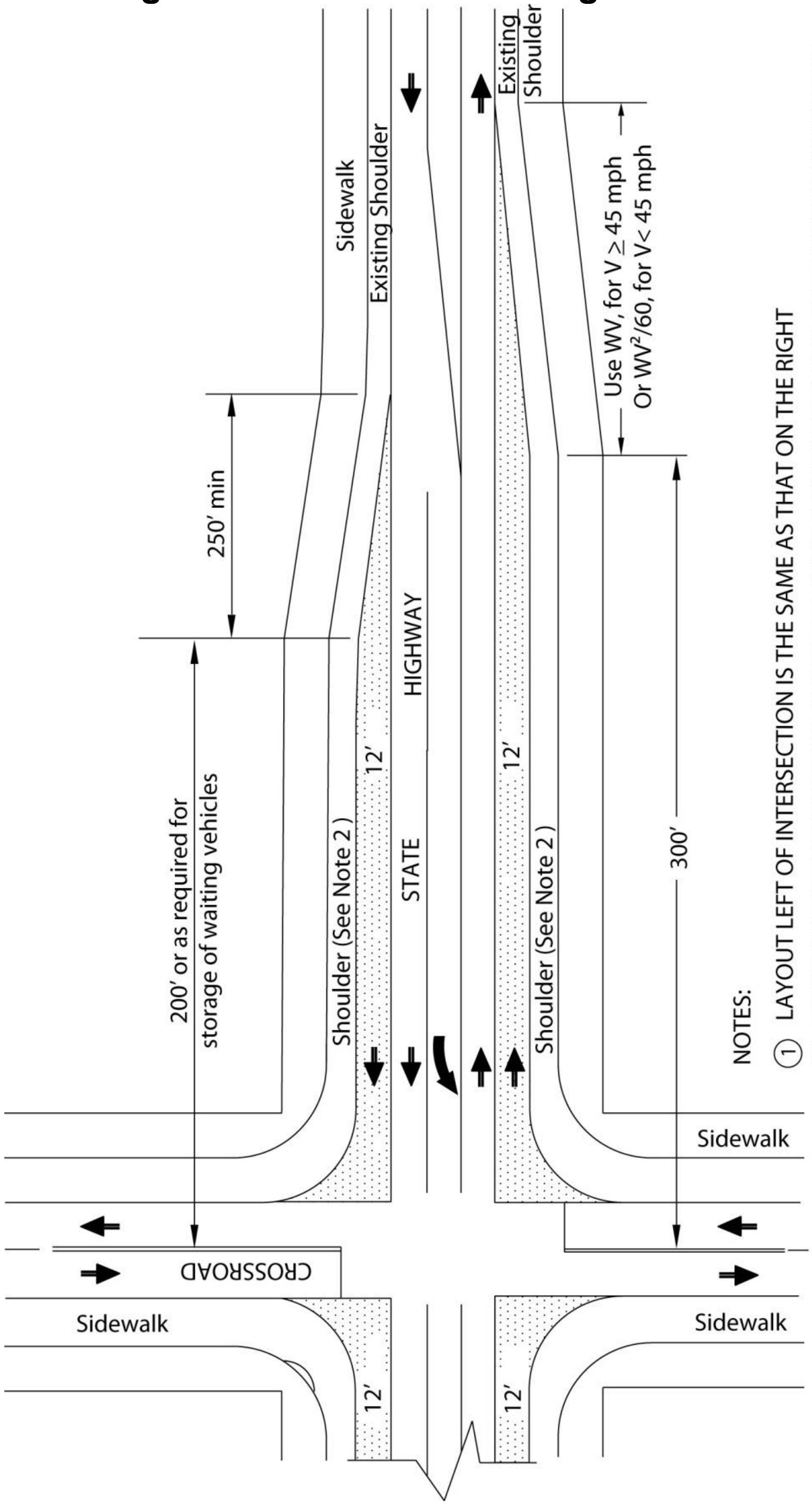
(2) Apply corner sight distance requirements at signalized intersections whenever possible due to unanticipated violations of the signals or malfunctions of the signals. See Index 405.1(2)(b).

(4) Acceleration Lanes for Turning Moves onto State Highways. At rural intersections, with "STOP" control on the local cross road, acceleration lanes for left and right turns onto the State facility should be considered. At a minimum, the following features should be evaluated for both the major highway and the cross road:

- divided versus undivided
- number of lanes
- design speed
- gradient
- lane, shoulder and median width
- traffic volume and composition of highway users, including trucks and transit vehicles

Figure 405.9

Widening of Two-lane Roads at Signalized Intersections



NOTES:

1. LAYOUT LEFT OF INTERSECTION IS THE SAME AS THAT ON THE RIGHT
2. WHERE WIDTH IS RESTRICTED SHOULDER WIDTH MAY BE REDUCED AND PARKING RESTRICTED WITH AN APPROVED DESIGN EXCEPTION PURSUANT TO INDEX 82.2.
3. FOR BICYCLE USE IN RURAL AREAS NON MAIN STREET PLACE TYPES, THE BIKE LANE IN THIS FIGURE IS PART OF THE SHOULDER. SEE INDEX 302.1 FOR FURTHER GUIDANCE.
4. CURB RAMPS NOT SHOWN. CURB RAMPS ARE TO BE PROVIDED PER DIB 82.

WIDENING

pattern of traffic movements. Unusual turning movement patterns may possibly call for a different shape of widening.

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

405.10 Roundabouts

Roundabout intersections on the State highway system must be developed and evaluated in accordance with National Cooperative Highway Research Program (NCHRP) Report 672 entitled “Roundabouts: An Informational Guide, Second Edition” dated October 2010 and Traffic Operations Policy Directive (TOPD) Number 13-02. Also see Index 401.5 for general information and guidance. See Figure 405.10A Roundabout Geometric Elements for nomenclature associated with roundabouts. Signs, striping and markings at roundabouts are to comply with the California MUTCD.

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

(a) Geometric design that reduce speeds at the entry and through the intersection, and

(b) The yield-at-entry rule, which requires traffic entering the intersection to yield to traffic that is traveling in the circulatory roadway.

Some benefits of roundabouts include:

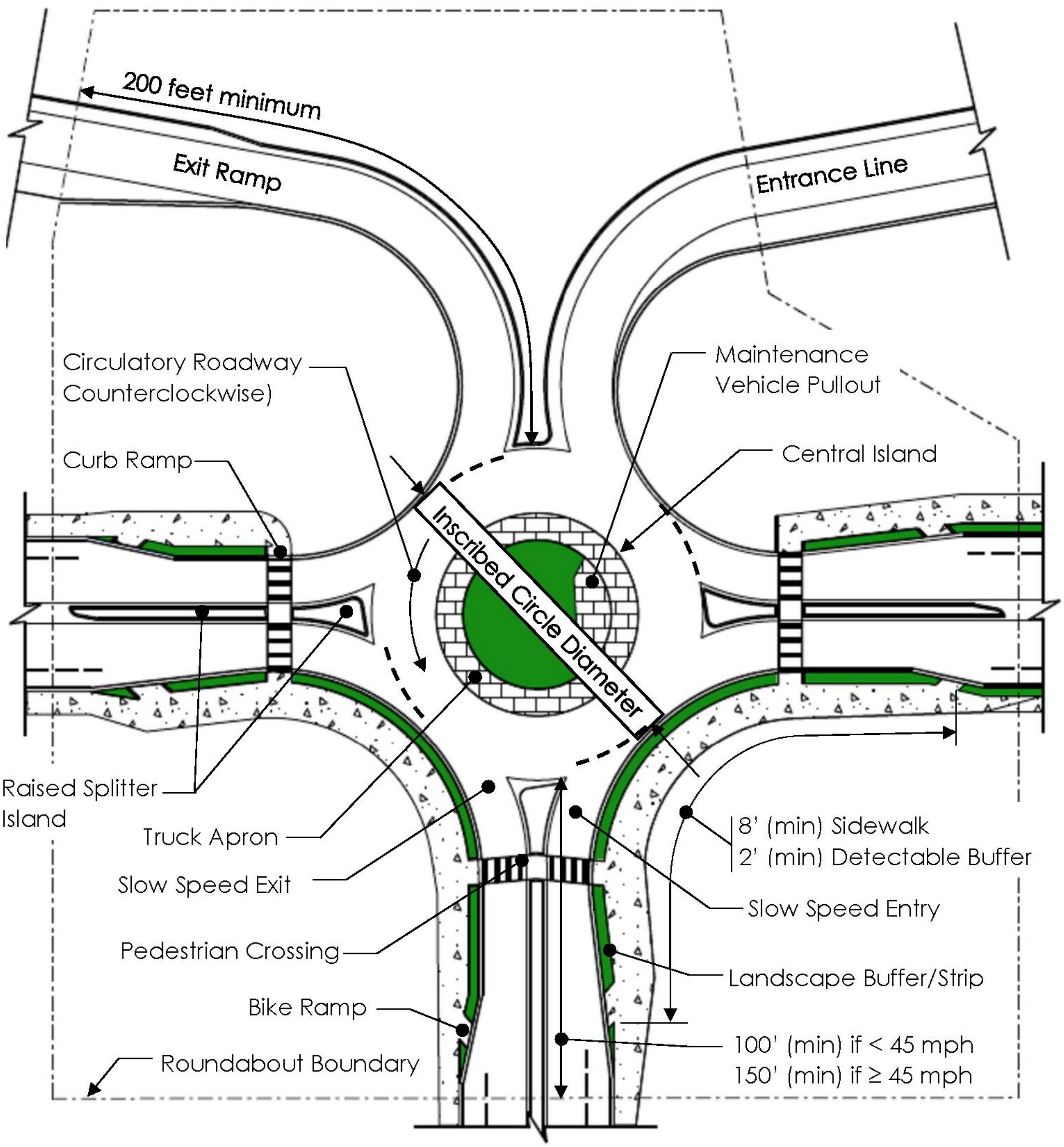
- Fewer conflict points, especially the high angle conflict points, which results in less severe crashes when compared to the stop-controlled or signal-controlled intersections. Over half of vehicle to vehicle conflict points associated with stop- and signal-controlled intersections are eliminated with the use of a roundabout. Additionally, a roundabout separates the conflict points which eases the ability of the driver, pedestrian, or bicyclist to identify a conflict and helps prevent conflicts from becoming crashes.
- Roundabouts are designed to reduce the vehicular speeds at intersections. Lower speeds lessen the vehicular crash severity. Likewise, studies indicate that when motorized vehicles are traveling at slower speeds, crash severity with pedestrian and bicyclist is significantly reduced; hence, roundabouts are proven safety countermeasures for traffic calming for complete street designs.
- Roundabouts are yield-controlled intersections, which allow continuous free flow of vehicles, pedestrians, and bicycles when no conflicts exist. This results in less noise and air pollution and reduces overall delays at roundabout intersections. Additionally, since there is no traffic signal, the operations are not affected by power outages.
- Roundabouts tend to have less delay and reduce greenhouse gases when compared to stop-controlled or signal controlled intersections.

Except as indicated in this Index, the design standards elsewhere in this manual do not apply within the boundaries of roundabouts. The boundary of a roundabout intersection is defined as follows:

- If the roadway is undivided, use the approach ends of the splitter islands.

Figure 405.10A

Roundabout Geometric Elements



NOTE:

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

- If the roadway is physically divided (such as a raised median), use 100 feet from the inscribed circle diameter (ICD) for roadways with posted speeds less than 45 miles per hour or 150 feet from the ICD for roadways with posted speeds equal to or greater than 45 miles per hour.
- For freeway exit ramps, a minimum of 200 feet from the ICD.
- For freeway entrance ramps, a minimum of 50 feet from the ICD or 20 feet past the crosswalk, whichever is further from the ICD.
- If successive curves are used, also known as a chicane (NCHRP Report 672, Exhibit 6-70), use the beginning of the chicane as the boundary.

See Figure 405.10A for a typical roundabout boundary example.

(1) *Design Period.* The 20-year design period required by Index 103.2 applies to roundabouts. Within the required design period, a single lane roundabout may provide operational benefits for 10 or more years before a multilane configuration is necessary, depending on traffic demand. In these situations, consideration should be given to build a single lane roundabout as an initial phase of a multilane roundabout. The multilane roundabout is designed, first, and needs to meet the performance checks for a multilane roundabout configuration. The single lane roundabout is designed from the multilane design by curbing out portions of the multilane roundabout to meet all the performance checks for a single lane configuration. This approach will maximize the safety and operational benefits during the initial and final phases of the improvements. In order to comply with the design period requirement, the initial project must provide the right of way needed for utility relocations, a shared-use path designed for a Class I Bikeway, and all other features other than pavement, lighting, and striping in their ultimate locations.

    Another method to meet the 20-year design period is to determine if metering one or more of the legs would allow a single lane roundabout to function for the life of the project.

    In some locations, it may not be practical to build a single lane roundabout that will operate for 10 years. Geometric constraints and other conflicts may preclude widening to the ultimate configuration. In such cases, other intersection configurations or control strategies addressed in Index 401.5 may need to be considered.

    When intersection improvements are to be phased, see NCHRP Report 672, Section 6.12.

(2) *Design Vehicles.* See Topic 404. Each roundabout has three design vehicles that need to be considered. 1) The largest vehicle to pass through the roundabout without using the truck apron. These would include, but not limited to, large vehicles transporting passengers with injuries, physical disabilities, the elderly, and transit vehicles. Other large vehicles included with this consideration are those where shifting of vehicle contents would be undesirable, such as fire engines and single-unit delivery vehicles. 2) California Legal or STAA trucks. These are discussed below in more detail. 3) Permit vehicles (oversize/overweight) that need to be able to pass through the roundabout. Accommodations for permit vehicles are further discussed below.

    The turning path for the design vehicle, dictates many of the roundabout dimensions, see Index 404.5. The design vehicle wheel tracking and swept width are to be used when designing all entries, exits, and the circulatory roadway (see Index 404.2). To maximize the safety benefits and preserve operations, roundabouts should be designed to be as small as possible and incorporate truck aprons to facilitate truck movements rather than increasing the size of the roundabout. Accommodations for trucks includes the use of truck aprons where necessary, while balancing other factors such as pedestrian accommodation.

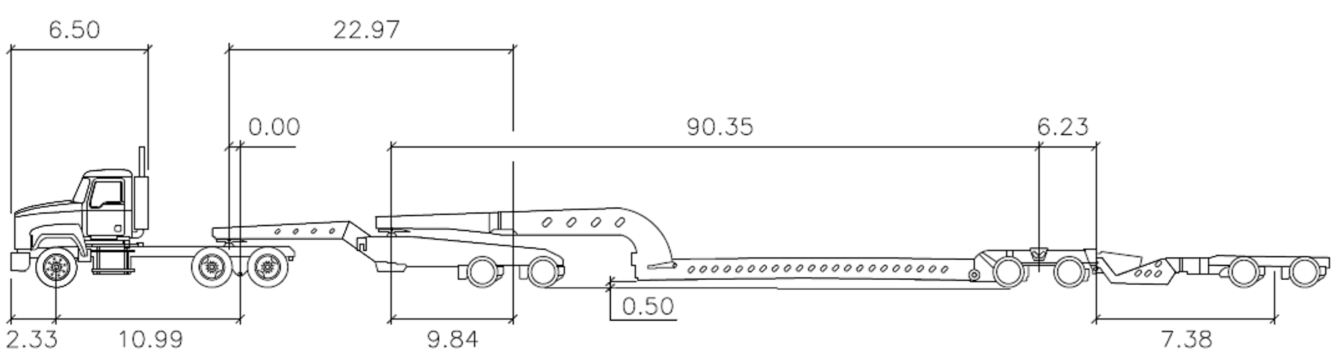
When more than one entry lane or circulating lane is available, truck drivers prefer to take control of the intersection by: 1) taking both lanes on entry to prevent a vehicle from entering their blind spots or overtaking during entry (as they would with a turn at any other intersection configuration); and 2) encroaching into adjacent lanes while circulating to protect their blind spots and prevent overtaking. During a turning movement, a truck's entire outside of the trailer (passenger side for left turns) becomes a large blind spot. To protect this blind spot and reduce the risk of colliding with an unseen vehicle, truck drivers may even prefer to stay in the outside lane while circulating to make through, left turn, and u-turn movements. Roundabouts with multilane entries or circulating lanes should be designed with the understanding that most truck drivers will not stay within the lane lines.

Oversize/Overweight permit vehicles are oftentimes larger than other design vehicles, but are essential to move goods throughout the state. Due to the critical nature of goods and services, which these vehicle transport, permit vehicles must also be considered in the design of intersections. Traditional signal or stop controlled intersections are typically not an issue for most of the oversize/overweight vehicles to pass through, but the circulatory roadway and central island of roundabouts can be an obstruction to get around. Roundabouts should not be overdesigned, nor should complete street elements be eliminated, for the occasional permit vehicle. Instead, truck aprons should be used to accommodate oversize/overweight vehicles, while preserving the speed control design of the roundabout. In addition to truck aprons, removable signs or other removable features in the central island or around the circular path can help ensure that these trucks can navigate the roundabout.

When designing for the oversize/overweight permit vehicle passing through the roundabout and staying on the state highway, a larger design vehicle is needed to design the roundabout entries, circulating roadway, and exits. The minimum design permit vehicle to be used is a "CALTRANS OS OW HEAVY HAUL" as designated in AutoTURN. See Figure 405.10B for this design vehicle. In addition to accommodating truck off-tracking, truck aprons at central and splitter islands, the design should accommodate an oversize/overweight vehicle with 6 inches of vertical clearance to pass through the intersection as shown in Figure 405.10B. A curb at the back of the central island truck apron, if needed, should not impede the oversize/overweight vehicle's movements. Oversize/overweight vehicles are not required to stay in their own lane.

Figure 405.10B

Oversize/Overweight Design Vehicle-CALTRANS OS OW HEAVY HAUL



The design permit vehicle as discussed may not be the appropriate permit vehicle for design purposes. For example, local situations such as farm areas where wide farm equipment or long components for windmills near a windmill power facility may necessitate a special vehicle design to pass through the roundabout. Therefore, the designer must coordinate with the District Truck Access Manager to ascertain the oversize/overweight vehicles that are to be accommodated and any additional vehicle requirements. Designers should also work with the District Truck Access Manager to determine if the permit vehicles will have an alternative route to bypass the roundabout or if they have to pass through this intersection.

To accurately simulate the design vehicle swept width traveling through a roundabout, the minimum speed of the design vehicle used in computer simulation software (e.g., Auto TURN) should be less than 5 miles per hour through the roundabout. Tracking and swept width must not encroach onto sidewalks or pedestrian refuge areas.

(3) *Inscribed Circle Diameter.* At single lane roundabouts, the size of the inscribed circle is largely dependent upon the turning requirements of the design vehicles. The inscribed circle diameter (ICD) must be large enough to accommodate: (a) the STAA design vehicle for all roundabouts on the National Network and on Terminal Access routes; and, (b) the California Legal design vehicle on all non-STAA route intersections on California Legal routes and California Legal KPRA Advisory routes, while maintaining adequate deflection curvature to ensure appropriate travel speeds for smaller vehicles. The non-permit design vehicle is to navigate the roundabout with the front tractor's wheels staying off the truck apron. The ICD for a single lane roundabout generally ranges between 105 feet to 150 feet to accommodate the California Legal design vehicle and 130 feet to 180 feet to accommodate the STAA design vehicle.

At multilane roundabouts, the ICD is to achieve adequate alignment of the natural vehicle path while maintaining deflection curvature to ensure appropriate travel speeds. To achieve both of these design objectives requires a slightly larger diameter than used for a single lane roundabout. The ICD for a multilane (2-lane) roundabout generally ranges between 150 feet to 220 feet to accommodate the California Legal design vehicle for non-STAA route intersections on California Legal routes and California Legal KPRA Advisory routes, and 165 feet to 220 feet to accommodate the STAA design vehicle for roundabouts on the National Network and on Terminal Access routes. Similar to a single lane roundabout, the non-permit design vehicle is to be able to navigate a multilane roundabout with the front tractor wheels staying off the truck apron.

A mini-roundabout may be an acceptable roundabout design on low volume routes or ramp terminals. With an inscribed circle diameter of less than 90 feet, the central island will need to be fully traversable to accommodate the design vehicles movements.

The ICD ranges given above are typical values; the design may be larger or smaller. Site location constraints and performance checks will determine if the diameter is appropriate for the location.

(4) *Entry Speeds.* Lowering the speed of vehicles entering and traveling through the roundabout is a primary design objective that is achieved by approach alignment and entry geometry.

<u>The following entry speeds should not be exceeded</u>:

- <u>Single lane entry, 25 miles per hour.</u>
- <u>Multilane entry, 30 miles per hour.</u>

A bypass lane is not included in the number of entry lanes. Although, a bypass lane prohibits entry into the circulatory roadway, the bypass lane should also meet the speed requirements to ensure that diverge and merge speeds are within 10 mile per hour of each other and vehicle speeds are not more than 20 mile per hours at pedestrian crossings. See Index 405.3(2)(b).

Entry speeds are to be determined through fastest path analysis. The fastest path is the smoothest, flattest path possible for a single vehicle in the absence of other traffic and ignoring all lane markings. The fastest path analysis should begin at least 165 feet from the ICD and should not bring the path closer than 3 feet from the edge of traveled way prior to the splitter island nor 5 feet from the face of a curb. These distances are minimums and the fastest path may occur further away from the curbs and striping depending on the roundabout configuration. For fastest path evaluation, see NCHRP Report 672, Section 6.7.1.

(5) *Exit Design.* Similar to entry design, exit design flexibility is required to achieve the optimal balance between competing design variables and project objectives to provide adequate capacity and safety while minimizing excessive property impacts and costs. Thus, the selection of a curved versus tangential design is to be based upon the balance of each of these criteria. Exit design is influenced by the place type, pedestrian demand, bicyclist needs, the design vehicle and physical constraints. The exit curb curve radii are usually larger than the entry curb curve radii in order to minimize the likelihood of congestion and crashes at the exits. Larger exit curb curve radii may also be needed to develop adequate deflection on the adjacent entry. However, the desire to minimize congestion at the exits needs to be balanced with the need to maintain an appropriate operating speed through the pedestrian crossing.

(6) *Number of Legs Serving the Roundabout.* Intersections with more than four legs are often difficult to manage operationally. Roundabouts are a proven traffic control device in such situations. However, it is necessary to ensure that the design vehicle can maneuver through all unrestricted legs of the roundabout. A public or private driveway as a leg to an intersection is not uncommon nor inappropriate and can be acceptable at non-interchange roundabouts as well. Driveways in and near proposed roundabouts should be included in the performance checks where applicable, including truck turning movements. For access control guidance at interchange ramp intersections, see Index 405.10 (13).

(7) *Sidewalk and shared use path.* The sidewalk should be designed as a shared use path, since the path will serve both pedestrians and those bicyclists who are not comfortable taking the lane to proceed though the roundabout. Although the sidewalk is considered a shared use path, it does not need to meet the design standards in Index 1003, but it should meet the design standards within Index 405.10.

(a) *Pedestrian Use.* Sidewalks around the circular roadway are to be designed in accordance with guidance in Design Information Bulletin (DIB) 82 Pedestrian Accessibility Guidelines for Highway Projects but must also be designed to allow low-speed bicycles to circulate around the roundabout. In addition to the guidance in DIB 82, the following applies:

i. The detectable warning surface (truncated domes) alert a pedestrian that they are about to enter cross traffic and is required on curb ramps. They are not to be used on a bike ramp.

ii. Truck aprons and mountable curbs are not to be placed in or adjacent to pedestrian crossing areas to avoid confusing pedestrians on where the crossing begins.

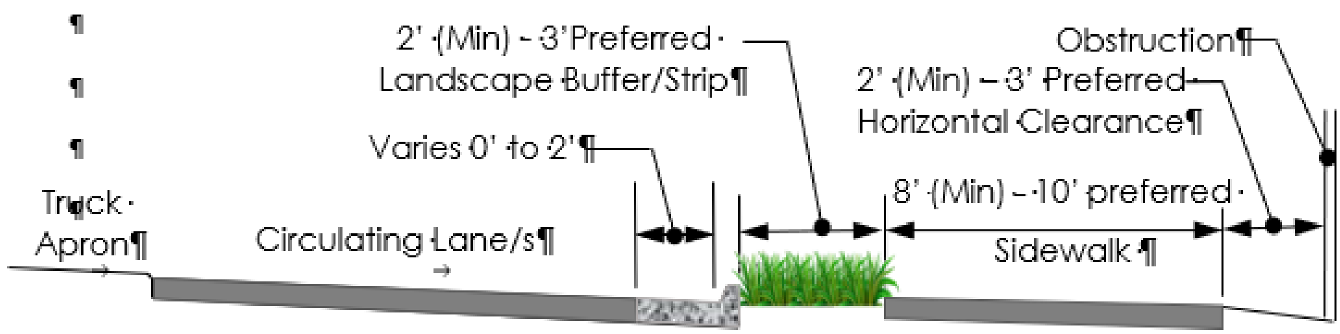
iii. See the California MUTCD for the signs and markings used at roundabouts.

iv. At pedestrian crossing locations the accessibility design will be treated as a midblock pedestrian street crossing. See DIB 82 for more information. Pedestrian crossings may also be used by bicyclists; thus, <u>these crossings need to be designed for both bicyclist and pedestrian needs and should be a minimum of 8 feet wide</u>, 10 feet is preferred.

v. A landscape buffer/strip, detectable by cane and under foot, between the sidewalk and the back of curb for the circular roadway of the roundabout should be a minimum of 2 feet wide, 3 feet is preferred. For more information see NCHRP Report 834, entitled "Crossing Solutions at Roundabout and Channelized Turn Lanes for Pedestrians with Vision Disabilities." See Figure 405.10C.

Figure 405.10C

Typical Cross Section of Buffer Between Sidewalk and Roundabout



(b) *Bicyclist Use.* Bicyclists may choose to travel in the circular roadway of a roundabout by taking a lane, while others may decide to travel using the sidewalk to bypass the circular roadway. Therefore, the approach and circular roadways, as well as the sidewalk, need to be designed for the mobility needs of bicyclists.

i. General. Bicycle ramps at roundabouts have the potential to be confused as pedestrian ramps, particularly for pedestrians who are visually impaired. The detectable warning surface (truncated domes) are not to be used on bike ramps.

ii. Bicyclist Use of the Circular Roadway. A bicyclist approaching from the roadway shoulder needs to be provided a choice of merging left into the lane to traverse the roundabout in the circulating lane with the vehicles or using the bicycle ramp to use the sidewalk. Single lane roundabouts tend to be comfortable for most bicyclists to travel through. Even, multilane roundabouts, that require bicyclists to change lanes at the entry to the circular roadway to select the appropriate lane for their direction of travel, appear to be comfortable for bicyclists who prefer to travel in the traveled way.

iii. Bicyclists Use of the Sidewalk. At roundabouts, bicyclists may ride around the roundabout using the sidewalk along with the pedestrians. This multi-modal situation introduces additional design considerations. The sidewalk is to be designed using the guidance in NCHRP Report 672 Section 6.8.2.2, the accessibility guidance in DIB 82, and the following:

- To accommodate mounted cyclists and pedestrians, a sidewalk should be included and have a minimum paved width of 8 feet, 10 feet is preferred.
- A minimum 2-foot horizontal clearance from the outside paved edge of the sidewalk to obstructions should be provided, 3 feet is preferred. Although not considered a shoulder, the surface should be flush with the edge of the sidewalk to allow bicyclists to recover. The surface material only needs to support the sidewalk pavement and minimize edge drop-off.

- Bicycle ramps are to be located to avoid being confused with curb ramps for pedestrians.
- The design details of the bicycle ramp are also important to the bicyclist. Bicycle ramps should be placed at a 35 to 45 degree angle to the departure roadway to enable the bicyclists to use the ramp while discouraging bicyclists from entering the sidewalk at a high speed.
- If there is a difference in the standards, the accessibility guidance in DIB 82 is to be followed to ensure the facility is accessible to pedestrians with disabilities.

(8) *Transit Use.* Transit vehicles and buses will not have difficulty negotiating a roundabout when it has been designed using the California Legal design vehicle or the STAA design vehicle. However, to minimize passenger discomfort, a roundabout should be designed such that the transit vehicle or bus does not use the truck apron.

(9) *Stopping Sight Distance and Visibility.* See NCHRP Report 672 Section 6.7.3 for stopping sight distance guidance at roundabouts.

A domed or mounded central island, between 3.5 to 6 feet high, is needed to focus attention on the approach and through roundabout alignment. A domed central island provides a visual screen to the downstream alignment and other distractions, and provides a visual cue for vehicles approaching the roundabout.

In high speed environments, additional lighting of, and vertical elements in the central island (i.e., landscaping and aesthetic features) may be needed.

(10) *Speed Consistency.* Consistency in operating speeds between the various movements within the roundabout can minimize collisions between traffic streams. The operating speeds between competing traffic streams and between consecutive geometric elements should be minimized such that the maximum speed differential between them is no more than 10 miles per hour.

(11) *Path Alignment (Natural Path).* As two traffic streams approach a multilane entry roundabout in adjacent lanes, drivers and bicyclists will be guided by lane markings up to the entrance line. At the yield point, they will continue along their natural trajectory into the circulatory roadway. The speed and orientation of the vehicle at the entrance line determines what can be described as its natural path. The geometry of the exits also affects the natural path that the vehicle travels. The natural path of two adjacent entering vehicles are not to overlap, see NCHRP Report 672, Section 6.7.2.

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

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

Roadways with posted speeds greater than or equal to 45 miles per hour, or at freeway exit ramps, require the splitter island lengths, as measured from the ICD, to be 200 feet. In some instances, a longer splitter island may be desirable. Concrete curb is to be provided on the right side of the approach roadway equal to the length of the splitter island.

At a low volume driveway leg of a roundabout, the splitter island may be eliminated if there would be no disruption to traffic flow.

(13)*Access Control.* The access control standards in Index 504.3(3) and 504.8 apply to roundabouts at interchange ramp intersections. The dimensions shown in Index 504.8 are to be measured from the ICD.

Driveways can occur within the roundabout or outside the access control. Driveways on approach and egress legs should not be placed within 50 feet from the ICD. Regarding crosswalk placement, consider the direction a driver exiting the driveway is looking for approaching traffic. Driveways should not be within 50 feet of a downstream crosswalk. At non-interchange roundabouts, a driveway may be added as a leg to a roundabout if supported operationally. Also consider if a driveway would be appropriate at that location if the intersection were eventually to become a signalized intersection.

(14)*Lighting.* Lighting is required at all roundabouts. See NCHRP Report 672 Chapter 8, the Traffic Manual Chapter 9 as well as consult with the District Traffic Safety Engineer.

(15)*Landscaping.* Landscaping should be designed such that drivers and bicyclists can observe the signing and shape of the roundabout as they approach, allowing adequate visibility for making decisions prior to and within the roundabout. The landscaping of the central island can enhance the intersection by making it a focal point, by promoting lower speeds and by breaking the headlight glare of oncoming vehicles or bicycles. It is desirable to create a domed or mounded central island, between 3.5 to 6 feet high, to increase the visibility of the intersection on the approach. Central island landscaping should provide elements that absorb kinetic energy of errant vehicles or redirect the vehicle, and not include features that may lead to the vehicle becoming airborne. Contact the District Landscape Architecture Unit to provide technical assistance in designing the roundabout landscaping. See Chapter 900 for additional Landscape Architecture requirements.

(16)*Vertical Clearance.* The vertical clearance guidance provided in Index 309.2 applies to roundabouts.

(17)*Drainage Design*. See Chapters 800 to 890 for further guidance.

(18)*Maintenance.* Contact the District Maintenance Engineer and appropriate Regional Manager for maintenance strategies and practices, including seasonal operations, maintenance resources, and specialized equipment. Maintenance responsibilities may also include multiple state, county, and city agencies where coordination of maintenance efforts and funding is needed.

Consider maintenance of the central island. Provide a maintenance vehicle pullout within the central island beyond the truck apron, so maintenance vehicles will not conflict with circulating trucks, see Figure 405.10A.

(19)*Snow Areas.* In climate regions where snowfall requires the use of snow removal equipment, consider the equipment to be used. Design ICD's as well as entrance and exit geometry to accommodate snow removal equipment and plow limitations. Check with District Maintenance for their requirements and limitations. Geometric elements to consider that facilitate snow removal are; mountable curb, tapering the ends of curbs down to allow plows to ride over curbs, plowing accommodation in both directions, providing snow storage space within the central island, and providing minimum entry/exit widths to accommodate the plow blade. Mountable curb may be used if the sidewalk/shared use path is not contiguous to the curb. Provide a planter or textured pavement between the path and the roadway. Snow storage areas must be designed to prevent snow melt from entering the circulating lanes where it can freeze. Snow storage areas must not block pedestrian paths.

(20) *Utilities.* Utility access openings (manholes) should not be located within the traveled way within the boundary of the roundabout. Roundabouts do not have shoulders to accommodate traffic while manholes are accessed. Manholes should not be allowed within the circulating roadway to avoid closing down the intersection during access. If a manhole is absolutely necessary within the boundary of the ICD, place it in the central island and off of the truck apron. Provide a maintenance vehicle pullout to allow access to the manhole without blocking truck traffic, see Figure 405.10A.

(21) *Performance Checks.* Roundabout design is not standard-based; it is a performance-based design method, which uses an iterative design process. As the design progresses through performance checks, as outlined in NCHRP Report 672, the geometrics of the design is refined. Any roundabout design submitted for geometric review without completed performance checks is an incomplete design. A roundabout design submitted for review during the Project Approval or Plan, Specification and Estimate phases must be accompanied by completed performance checks. However, a roundabout design submitted for review during the Project Initiation phase does not require completed performance checks.

(22) Turbo Roundabouts. Some multilane roundabouts experience higher than expected frequencies of sideswipe collisions. To prevent this, some countries have implemented a modified version of a multilane roundabout called the turbo roundabout. The significant differences between turbo roundabouts and modern roundabout are that a physical separation is included to prevent lane changes within the circulating lanes of the turbo roundabout. This separation results in a larger roundabout footprint with little to no deflection on entry and with all vehicles staying completely within their lane. To date there are only a few hundred worldwide and less than five in the US and Canada combined; consequently, there is limited information regarding the design of turbo roundabouts, especially regarding North American driver and vehicles. The traditional capacity analysis tools for roundabouts do not apply to turbo roundabouts. Due to the limited experience in the U.S. and lack of analysis tools, caution should be used before selecting a turbo roundabout for an intersection alternative on the State highway system.

Turbo roundabout intersections on the State highway system should be developed and evaluated in accordance with the following: The FHWA Safety Program Informational Primer entitled “Turbo Roundabouts;” NCHRP Report 672 “Roundabouts: An Informational Guide;” “Roundabouts-Application and Design, A Practical Manual,” from the Dutch Ministry of Transport, Public Works and Water Management, Partners for Roads; and, TOPD Number 13-02 “Intersection Control Evaluation.” Per FHWA’s recommendation for capacity analysis of turbo roundabouts, refer to Transportation Research Record (TRR) Issue Number 2517-08 entitled “Capacity Estimation on Turbo roundabouts with Gap Acceptance and Flow Level Methods.” Also see Index 401.5 for general information and guidance.

Topic 406 – Ramp Intersection Capacity Analysis

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

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

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

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

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

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

Table 406

Vehicle Traffic Flow Conditions at Intersections at Various Levels of Operation

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

NOTE:

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

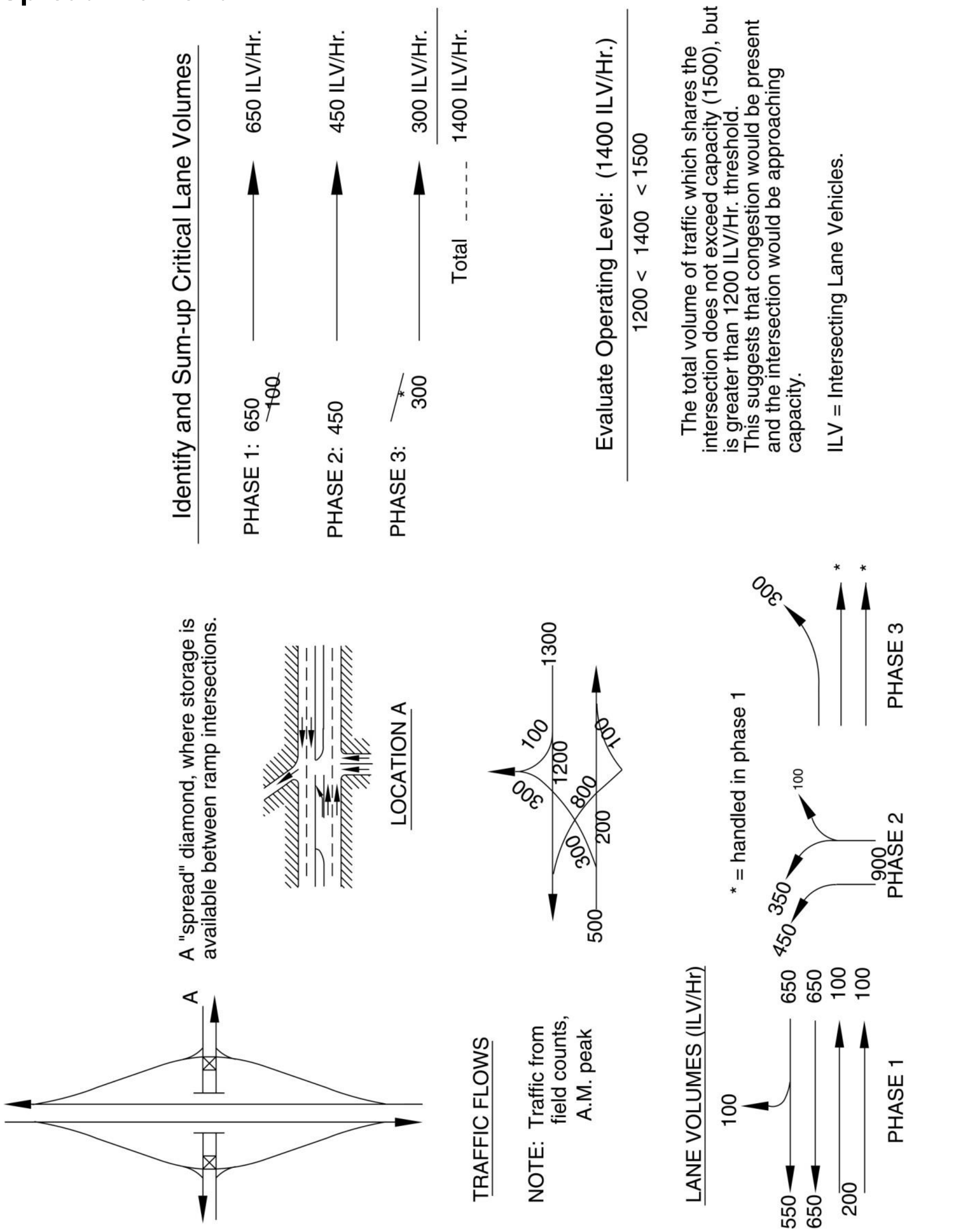
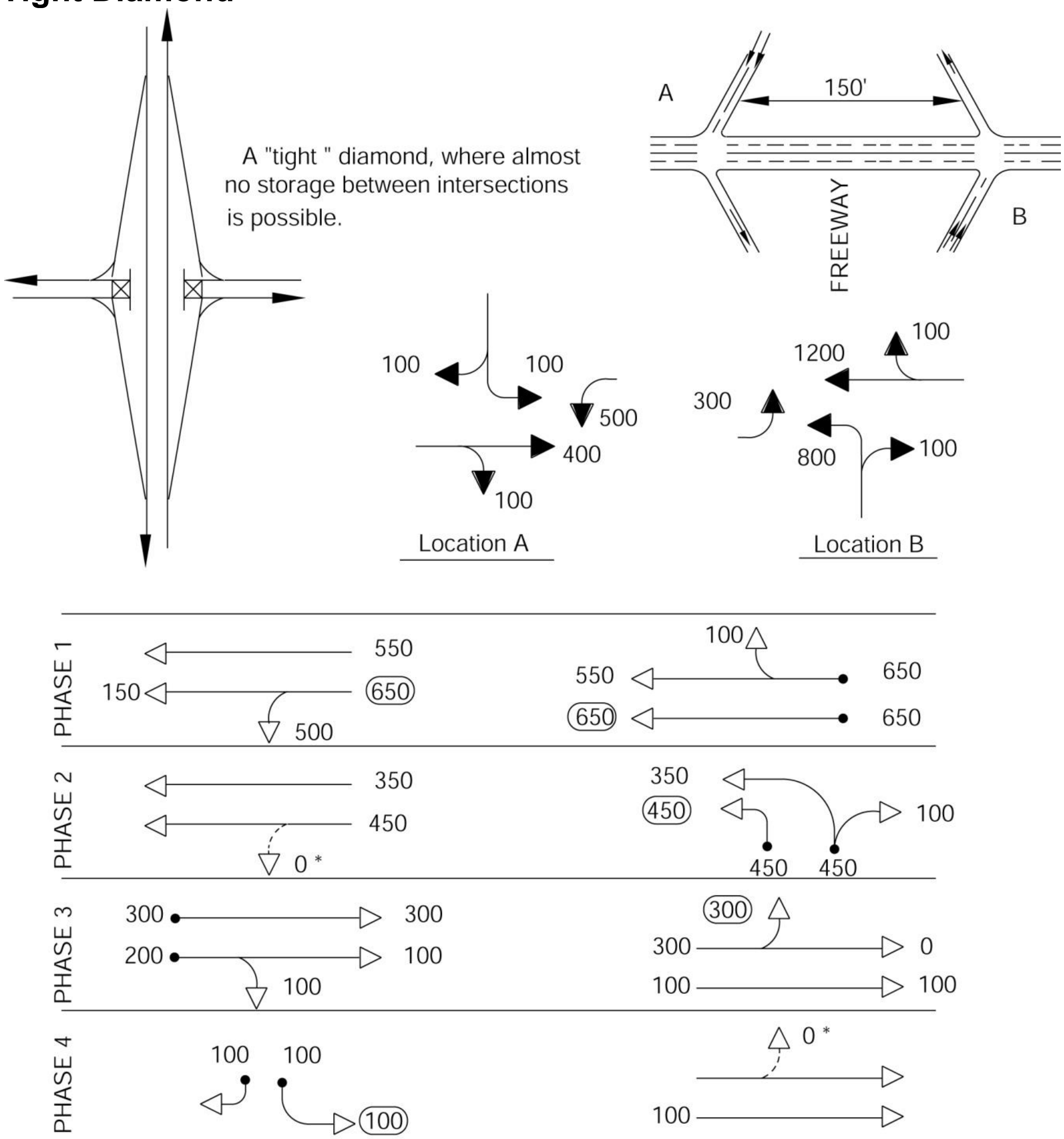


Figure 406B

Tight Diamond

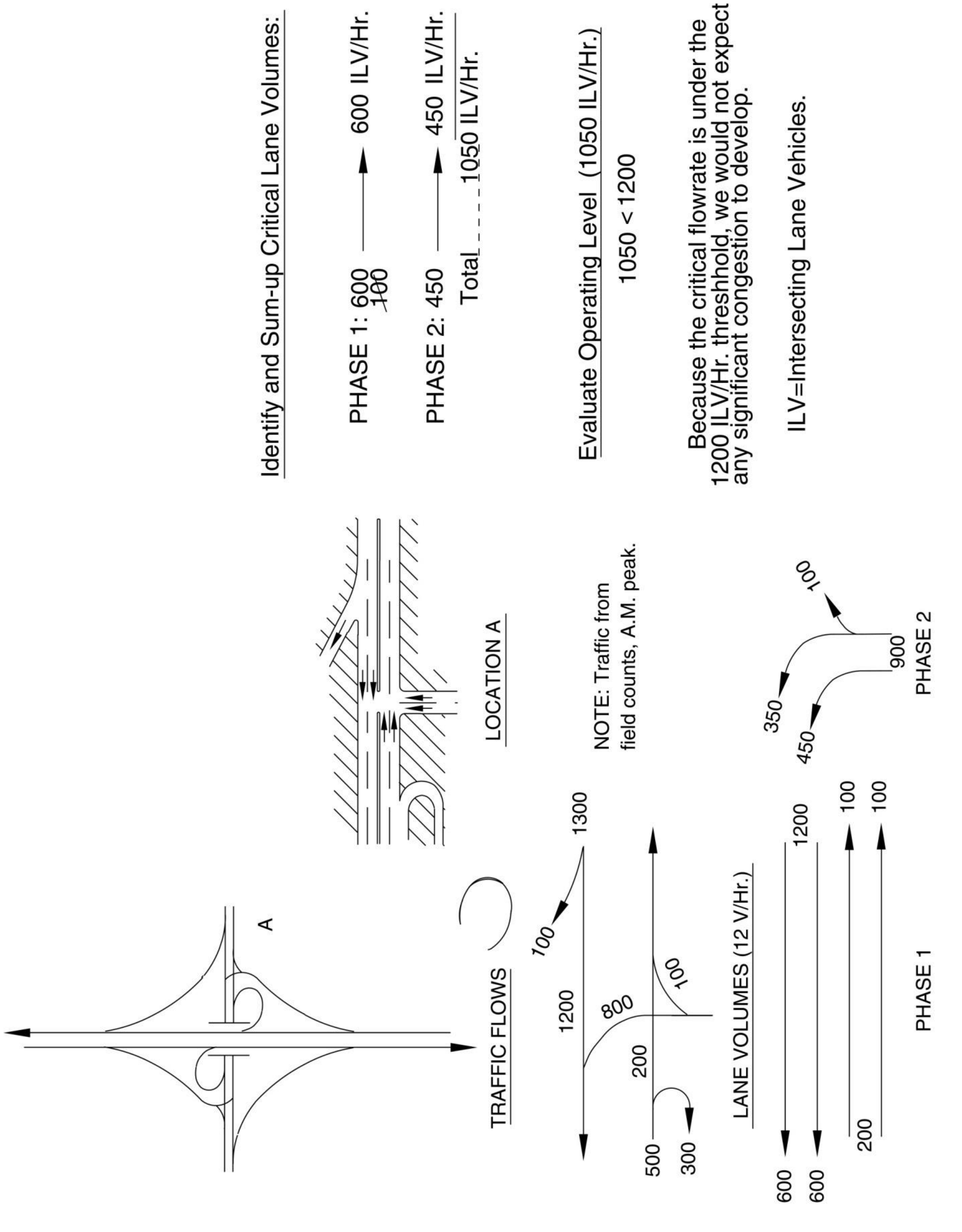


*NOTE: When no storage at all is permitted, left-turn movement is cleared during this phase.

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

Figure 406C

Two-quadrant Cloverleaf



CHAPTER 500 – TRAFFIC INTERCHANGES

Topic 501 – General

Index 501.1 – Concepts

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

501.2 Warrants

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

501.3 Spacing

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

The standards contained within this Index apply to:

- New interchanges.
- Modifications or replacement of an existing interchange structure that results in a change to the centerline measurement of crossroad-to-crossroad spacing.
- Projects to increase mainline capacity when existing interchanges do not meet interchange spacing requirements.

See Index 504.7 for additional technical requirements related to interchange spacing, based on entrance ramp-to-exit ramp spacing.

Topic 502 – Interchange Types

502.1 General

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

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

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

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

502.2 Local Street Interchanges

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

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

Table 504.3

Ramp Widening for Trucks

Ramp Radius (ft)	Widening (ft)	Lane Width (ft)
<150	8	20
150 – 179	5	17
180 – 209	4	16
210 – 249	3	15
250 – 299	2	14
-300 – 350	1	13
>350	0	12

(c) Shoulder Width. **Shoulder widths for ramps shall be as indicated in Table 302.1.** Typical ramp shoulder widths are 4 feet on the left and 8 feet on the right.

(d) Lane Drops. Typically, lane drops are to be accomplished over a distance equal to WV. Where ramps are metered, the recommended lane drop taper past the meter limit line is 50 to 1 (longitudinal to lateral). <u>Depending on approach geometry and speed, the lane drop transition between the limit line and the 6-foot separation point should be accomplished with a taper of between 30:1 and 50:1 (longitudinal to lateral).</u> This is further explained in Index 504.3(2)(b) for metered multilane entrance ramps. **However, the lane drop taper past the limit line shall not be less than 15 to 1.**

<u>Lane drop tapers should not extend beyond the 6-foot point without the provision of an auxiliary lane.</u>

(e) Lane Additions Lane additions to ramps are usually accomplished by use of a 120-foot bay taper. See Table 405.2A for the geometrics of bay tapers.

(2) Ramp Metering.

Caltrans Deputy Directive (DD) No. 35-R1, Ramp Metering, contains the statewide policy for ramp metering which delegates responsibility for its implementation in part through the Ramp Metering Design Manual (RMDM). DD 35-R1 specifies that provisions for entrance ramp metering shall be included in any project that proposes additional capacity, modification of an existing interchange, or construction of a new interchange, within the freeway corridors identified in the Ramp Metering Development Plan (RMDP), regardless of funding source. Projects designed for new or existing freeway segments experiencing recurring traffic congestion and/or a high frequency of vehicle collisions may include provisions for entrance ramp metering, whether or not the freeway segment locations are listed in the RMDP.

All geometric designs for ramp metering installations must be discussed with the Project Delivery Coordinator or District Design Liaison. Design features or elements which deviate from design standards require the approvals described in Index 82.2.

See the RMDM for ramp metering guidance, procedures, and policies to be used in conjunction with the guidance in this manual. Where traffic-related ramp metering guidance is noted in this Chapter, reference is made to the RMDM for exception instructions and further information. The number of lanes at the limit line denotes a metered single or multilane entrance ramp configuration.

Geometric ramp design for operational improvement projects which include ramp metering should be based on current peak-hour traffic volume. If this current data is not available it should be obtained before proceeding with design. Peak hour traffic data from the annual Caltrans Traffic Volumes book is not adequate for this application.

The design advice and typical designs that follow should not be directly applied to ramp meter installation projects, especially retrofit designs. Every effort should be made by the designer to exceed the recommended minimum standards provided herein, where conditions are not restrictive.

(a) Metered Freeway Entrance Ramps (1 General Purpose (GP) or 1 GP + 1 HOV Preferential Lane).

According to the RMDM, a High-Occupancy Vehicle (HOV) preferential lane shall be provided where ramp meters are installed, and each HOV preferential lane should be metered. See the RMDM for exception procedures from the Ramp Metering policy. See Figures 504.3A and 504.3B for typical freeway entrance ramp metering (1 GP Lane + 1 HOV Preferential Lane).

Due to the operational benefits of an auxiliary lane, metered single or multilane freeway entrance ramps should include an auxiliary lane with a minimum length of 300 feet downstream of the gore point. See Figures 504.3A and 504.3C.

Where truck (3 or more axles) volume is 5 percent or greater on ascending metered single or multilane freeway entrance ramps and connectors with sustained upgrades exceeding 3 percent at least throughout the merge area, a minimum 1,000-foot length of auxiliary lane should be provided downstream of the gore point

When vehicle volume exceeds 1500 vph, a 1,000 - foot minimum length of auxiliary lane should be provided downstream of the gore point for metered single or multilane freeway entrance ramps and connectors. If an auxiliary lane is present, the lane drop transition zone may extend to the gore point. However, the proximity of the nearest interchange may warrant weaving analysis to determine the acceptability of extending the ramp lane transition beyond the 6-foot separation point. A longer auxiliary lane should be considered where mainline/ramp gradients and truck volumes warrant additional length.

(b) HOV Preferential Lane.

Ramp meter installations should operate in conjunction with, and complement, other transportation management system elements and transportation modes. As such, ramp meter installations should include preferential treatment of carpools and transit riders. Specific treatment(s) must be tailored to the unique conditions at each ramp location.

Where restrictive conditions, vehicle volumes less than 500 vehicles per hour (vph), or other engineering judgement exist in support of an exception to the HOV preferential lane, see Figures 504.3C and 504.3D. Where truck (3 or more axles) volumes are 5 percent or greater on ascending metered single-lane freeway entrance ramps and connectors with sustained upgrades exceeding 3 percent at least throughout the merge area, a minimum 500-foot length of auxiliary lane should be provided downstream of the gore point.

Figure 504.3A

Typical Freeway Entrance Loop Ramp Metering (1 GP Lane + 1 HOV Preferential Lane)

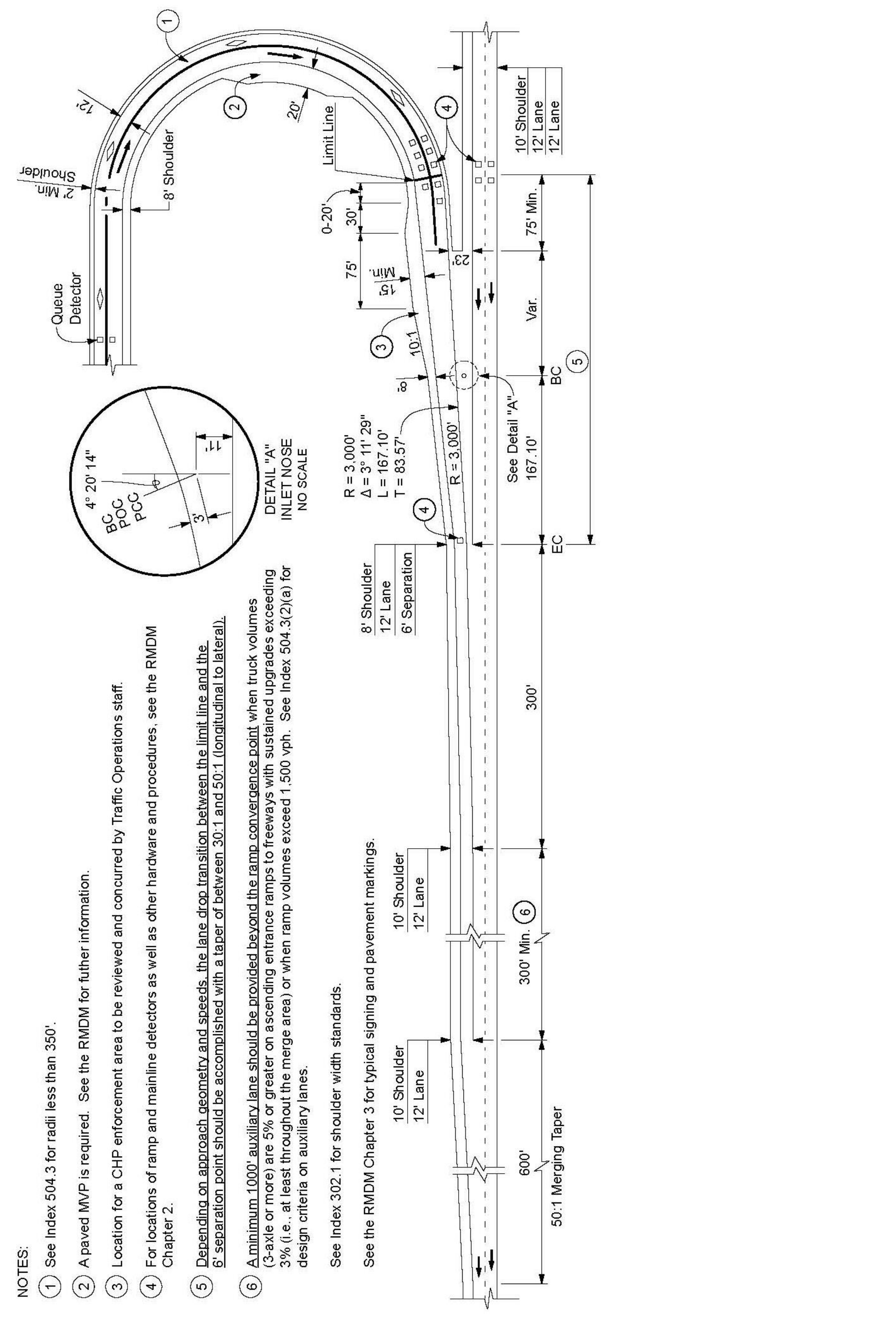
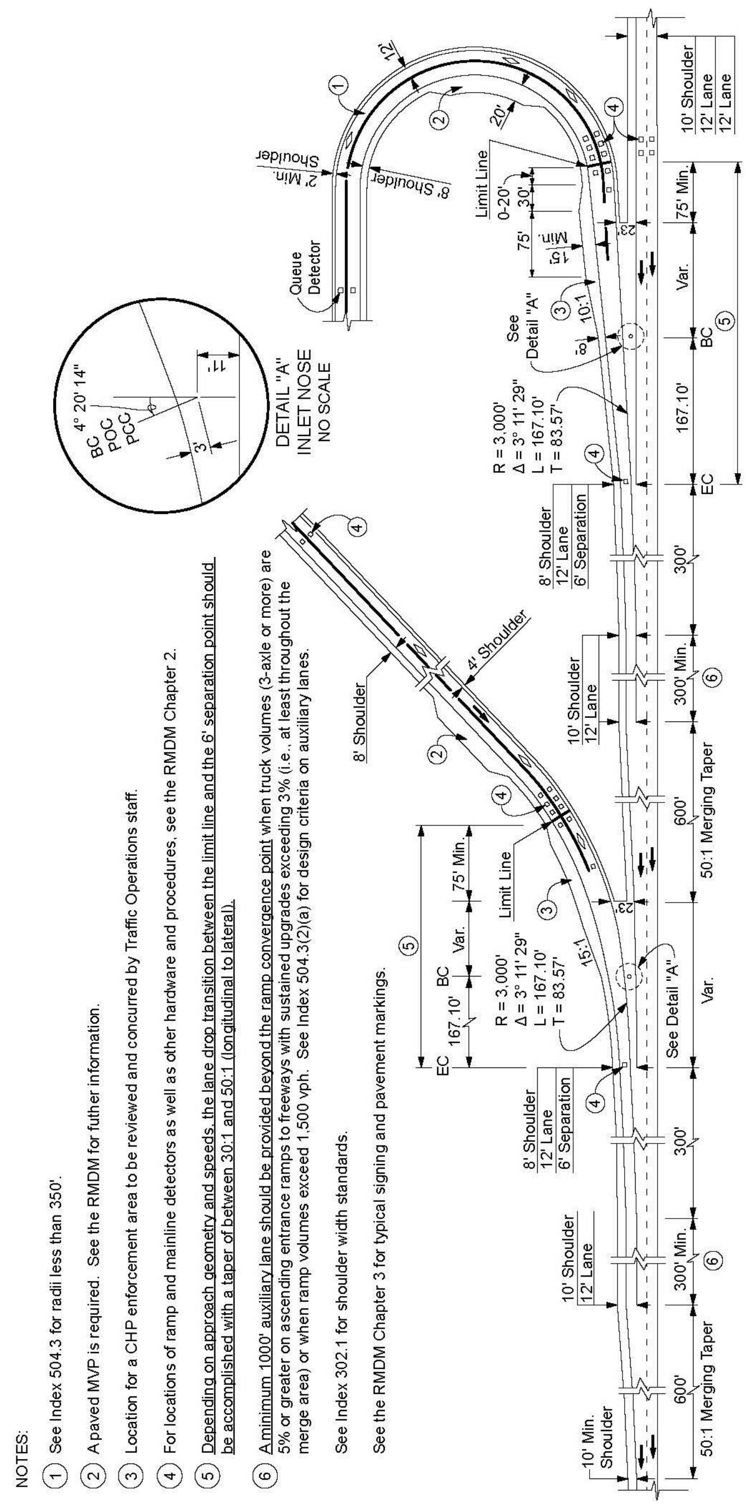


Figure 504.3B

Typical Successive Freeway Entrance Ramp Metering (1 GP Lane + 1 HOV Preferential Lane)



In general, the vehicle occupancy requirement for ramp meter HOV preferential lanes is typically two or more persons per vehicle. At some locations, a higher vehicle occupancy requirement may be necessary. The occupancy requirement should be based on the HOV demand and should match with other HOV facilities in the vicinity.

A HOV preferential lane should typically be placed on the left; however, demand and operational characteristics at the ramp entrance may dictate otherwise. Design of the HOV preferential lane at a metered entrance ramp requires the review and concurrence of the Caltrans District Traffic Operations Branch responsible for ramp metering.

Access to the HOV preferential lane may be provided in a variety of ways depending on interchange type and available storage length for queued vehicles. Where queued vehicles in the general purpose (GP) lane may block access to the HOV preferential lane, consider providing direct or separate access. To avoid trapping GP traffic in an HOV preferential lane, the signing and pavement marking at the ramp entrance should direct motorists into the GP lane(s). See the RMDM, Chapter 3 for signing and pavement markings. Designs should consider pedestrian/bicycle volumes, especially when the entrance ramp is located near a school or the local highway facility includes a designated bicycle lane or route. See Index 403.6 for right-turn-only lane guidance where bicycle travel is permitted. Contact the District Traffic Safety Engineer or designee and the Project Delivery Coordinator or District Design Liaison to discuss the application of specific design and/or general issues related to the design of HOV preferential lane access.

Signing for a HOV preferential lane should be placed to clearly indicate which lane is designated for HOVs. Real-time signing at the ramp entrance, such as an overhead changeable message sign, may be necessary at some locations if pavement delineation and normal signing do not provide drivers with adequate lane usage information. To avoid leading Single-Occupancy Vehicles (SOV) into a HOV preferential lane, pavement delineation at the ramp entrance should lead drivers into the SOV lane.

(c) Metered Multilane Freeway Entrance Ramps (2 GP + 1 HOV Preferential Lane).

The number of metered lanes at an entrance ramp is the number of both metered general purpose (GP) and high-occupancy vehicle (HOV) preferential lanes at the limit line. The minimum number of metered GP lanes is determined based on GP traffic demand. The number of metered HOV preferential lanes is determined based on HOV demand using the same guidelines as GP traffic demand, as well as the HOV preferential lane policy.

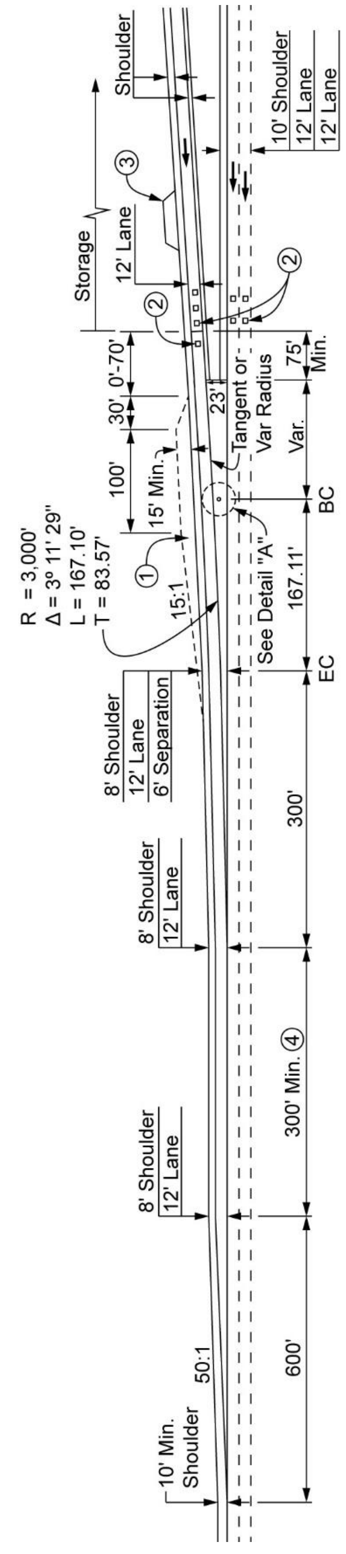
A multilane ramp segment may be provided to increase vehicle storage within the available ramp length. At on-ramps with peak hour volume between 500 and 900, a two-lane ramp meter may be provided to double the vehicles stored within the available storage area. See RMDM for additional multilane freeway entrance ramp guidance.

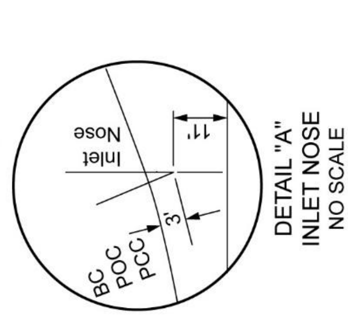
Figures 504.3E and 504.3F illustrate typical designs for metered multilane diagonal and loop freeway entrance ramps. On multilane loop ramps, typically only the right lane needs to be widened to accommodate design vehicle off-tracking. See Index 504.3(1)(b).

Three-lane metered ramps are typically needed to serve peak (i.e., commute) hour traffic along urban and suburban freeway corridors. The adverse effects of bus and truck traffic on the operation of these ramps (i.e., off-tracking, sight restriction, acceleration characteristics on upgrades, etc.) is minimized when the ramp alignment is tangential or consists of curve radii not less 300 feet. Proposed three-lane loop and four-lane entrance ramps require the review and approval by the Deputy District Director of Traffic Operations.

Figure 504.3C

Restrictive Condition Freeway Entrance Ramp Metering (1 GP Lane)





NOTES:

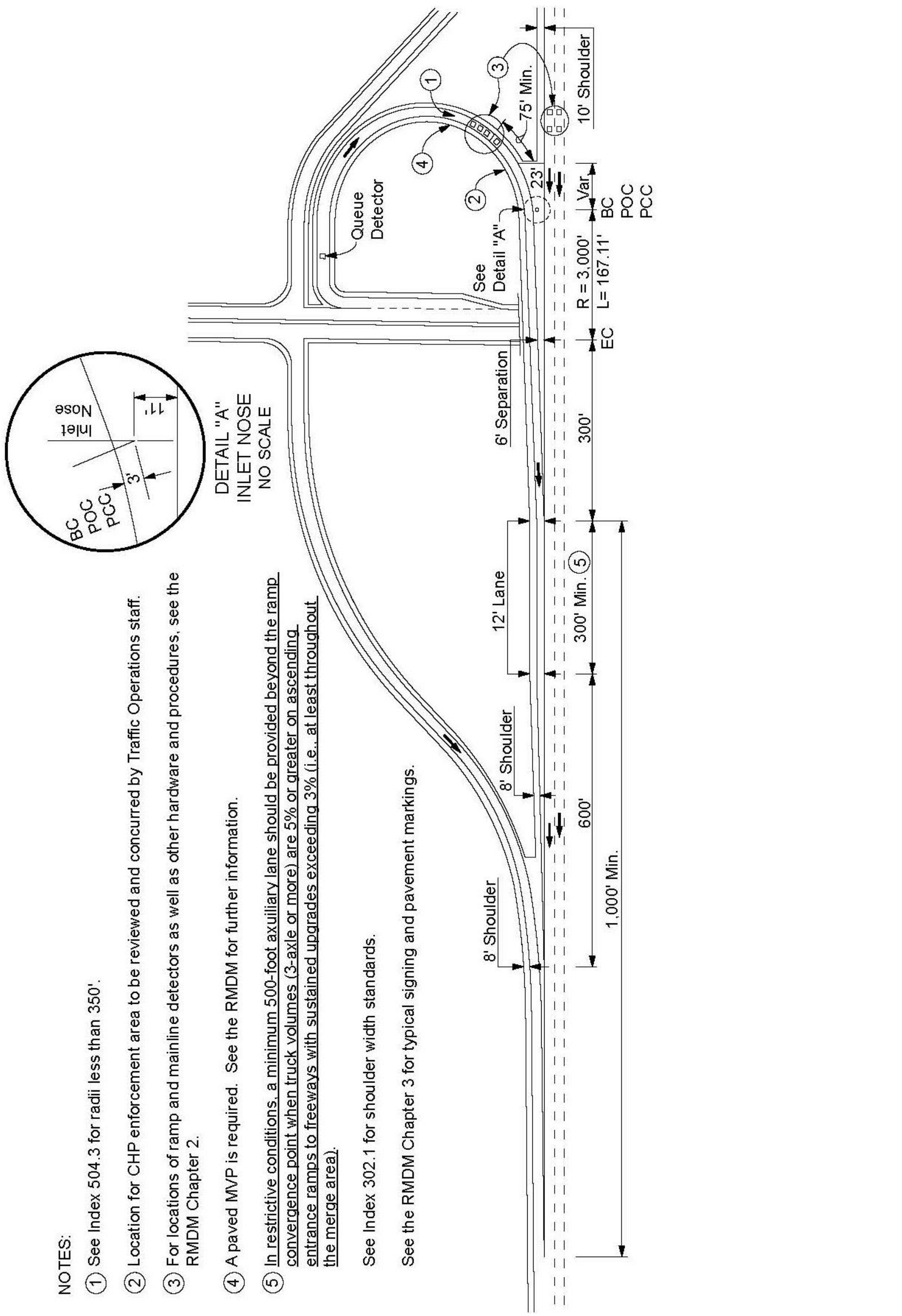
1. Location for CHP enforcement area to be reviewed and concurred by Traffic Operations staff.
2. For locations of ramp and mainline detectors as well as other hardware and procedures, see the RMDM Chapter 2.
3. A paved MVP is required. See the RMDM for further information.
4. In restrictive conditions, a minimum 500-foot axuiliary lane should be provided beyond the ramp convergence point when truck volumes (3-axle or more) are 5% or greater on ascending entrance ramps to freeways with sustained upgrades exceeding 3% (i.e., at least throughout the merge area).

See Index 302.1 for shoulder width standards.

See the RMDM Chapter 3 for typical signing and pavement markings.

Figure 504.3D

Restrictive Condition Freeway Entrance Loop Ramp Metering (1 GP Lane)



NOTES:

1. See Index 504.3 for radii less than 350'.
2. Location for CHP enforcement area to be reviewed and concurred by Traffic Operations staff.
3. For locations of ramp and mainline detectors as well as other hardware and procedures, see the RMDM Chapter 2.
4. A paved MVP is required. See the RMDM for further information.
5. In restrictive conditions, a minimum 500-foot auxiliary lane should be provided beyond the ramp convergence point when truck volumes (3-axle or more) are 5% or greater on ascending entrance ramps to freeways with sustained upgrades exceeding 3% (i.e., at least throughout the merge area).

See Index 302.1 for shoulder width standards.

See the RMDM Chapter 3 for typical signing and pavement markings.

The multilane segment of metered freeway entrance ramps and connectors should transition to a single lane width between the meter limit line and the 6-foot separation point.

The lane drop transition should be accomplished with a taper of 50:1 (longitudinal to lateral) unless a lesser taper is warranted by site and/or project specific conditions which control the ramp geometry and/or anticipated maximum speed of ramp traffic. For example, "loop" entrance ramps would normally not allow traffic to attain speeds which would warrant a 50:1 (longitudinal to lateral) lane drop taper. Also, in retrofit situations, existing physical, environmental or right of way constraints may make it impractical to provide a 50:1 taper, especially if the maximum anticipated approach speed will be less than 50 miles per hour. Depending on approach geometry and speed, (See Index 206.3 for how to match the vehicle speed to lane-drop taper ratio) the lane drop transition zone between the meter limit line and the 6-foot separation point of metered multilane freeway entrance ramps should be accomplished with a taper ratio of between 50:1 and 30:1 (longitudinal traveled way length to transverse lane width). **However, the lane drop taper ratio past the meter limit line for metered freeway multilane entrance ramps shall not be less than 15:1 (longitudinal traveled-way length to transverse lane width).**

The merge from the metered entrance ramp to the freeway should include a 300-foot minimum auxiliary lane beyond the ramp convergence point.

Where truck volumes (3-axle or more) are 5 percent or greater on ascending entrance ramps to freeways with sustained upgrades exceeding 3 percent (i.e. at least throughout the merge area), a minimum 1,000 feet length of auxiliary lane should be provided beyond the ramp convergence point. AASHTO, A Policy on Geometric Design of Highways and Streets, provides additional guidance on acceleration lane length on grades.

When ramp volumes exceed 1,500 vph, a 1,000-foot minimum length of auxiliary lane should be provided beyond the ramp convergence point. If an auxiliary lane is included, the ramp lane transition may be extended to the convergence point. However, the proximity of the nearest interchange may warrant weaving analysis to determine the acceptability of extending the ramp lane transition beyond the 6-foot separation point. A longer auxiliary lane should be considered where mainline/ramp gradients and truck volumes warrant additional length.

(d) Metered Freeway-to-Freeway Connectors.

Freeway-to-freeway connectors may also be metered. The need to meter a freeway-to-freeway connector should be determined on an individual basis. Because connector ramps provide a link between two high speed facilities, drivers do not expect to stop, nor do they expect to approach a stopped vehicle.

The installation of metering on connectors shall be limited to those facilities that meet or exceed the following geometric design criteria:

- **Standard connector lane and shoulder widths.**
- **"Taillight" sight distance, measured from a 3 ½-foot eye height to a 2-foot object height, is provided for a minimum design speed of 50 miles per hour.**

All lane drops on connectors should be accomplished over a distance not less than WV. **All lane drop transitions on connectors shall be accomplished with a taper of 50:1 (longitudinal traveled-way length to transverse lane width) minimum**, (see Figures 504.3G and 504.3H). See RMDM Section 1.11 for additional guidance.

Figure 504.3E

Typical Multilane Freeway Diagonal Entrance Ramp Metering (2 GP Lanes + 1 HOV Preferential Lane)

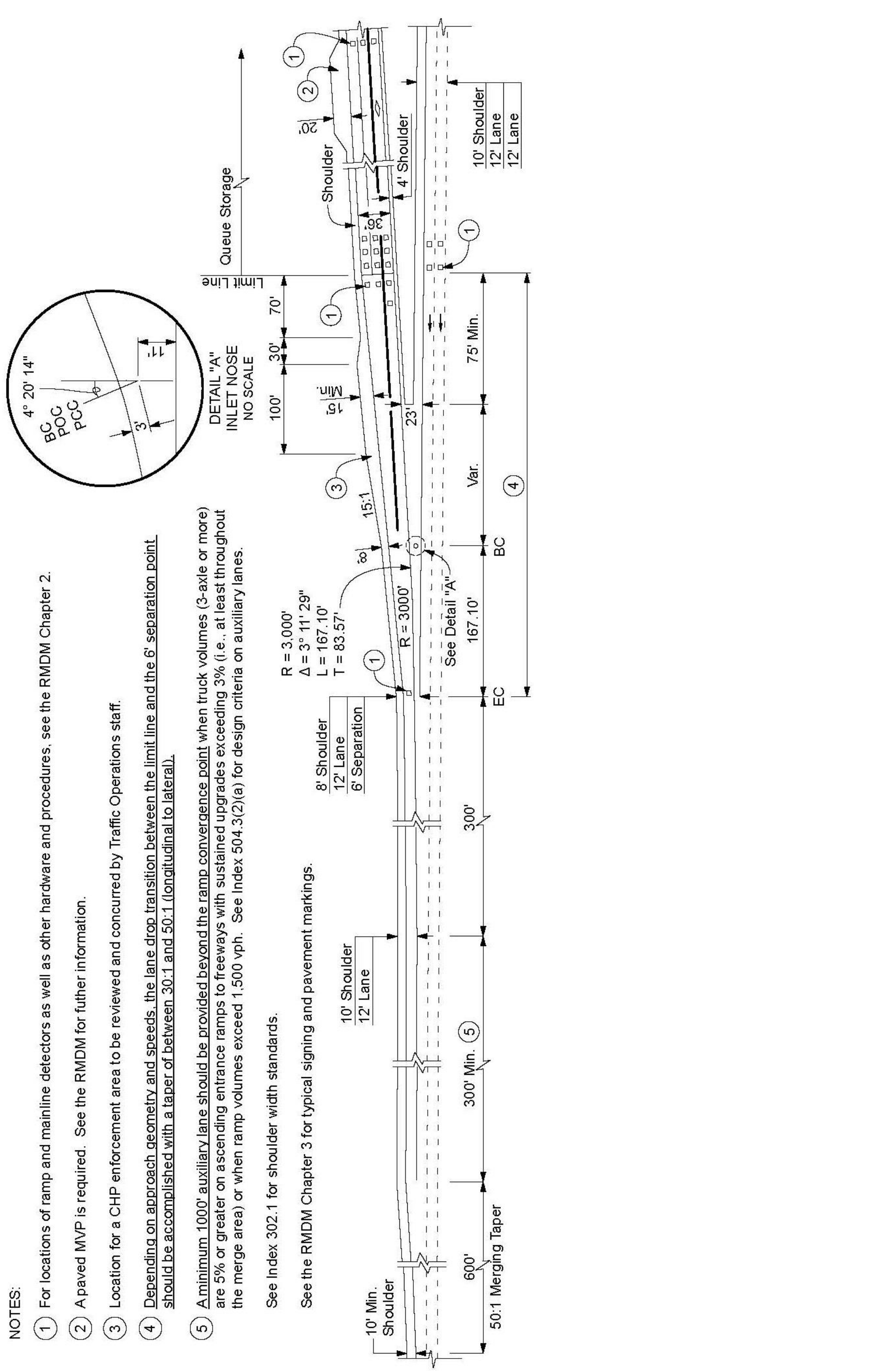
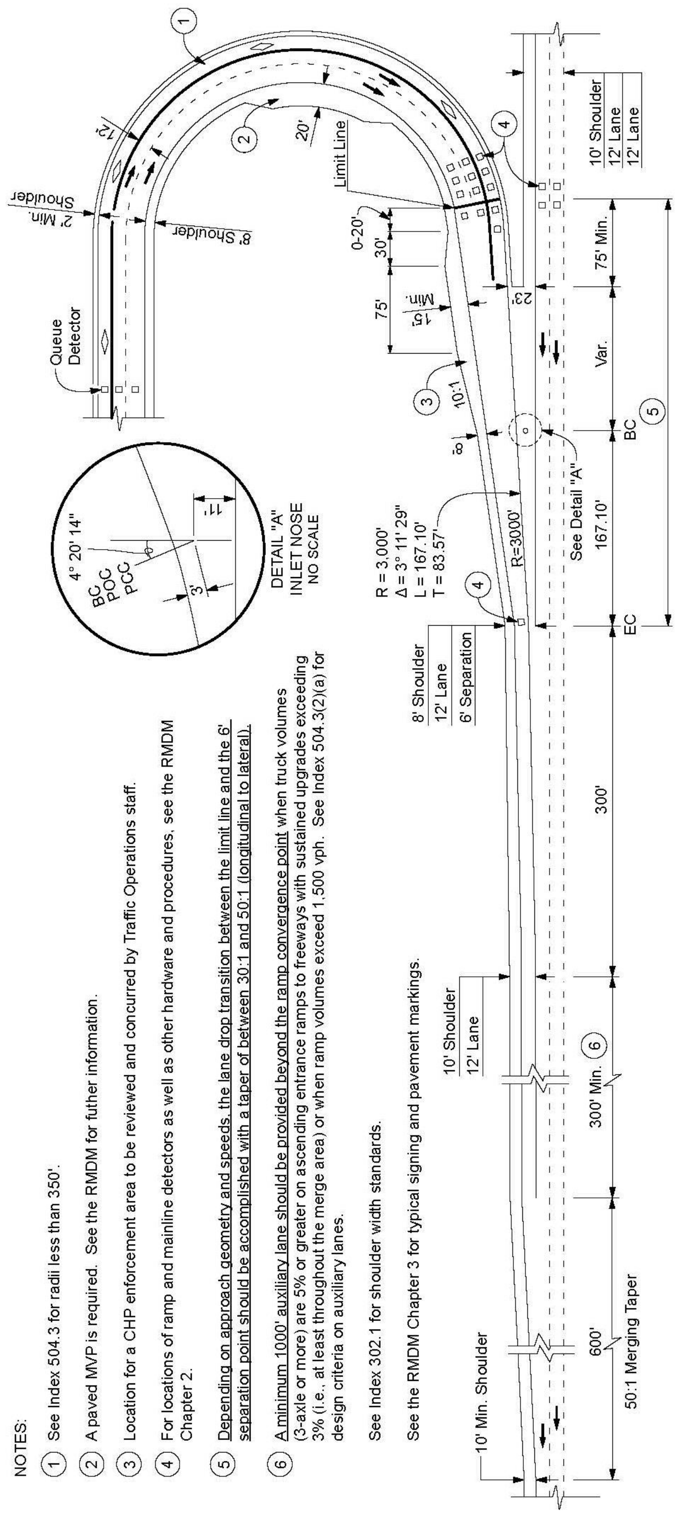


Figure 504.3F

Typical Multilane Freeway Loop Entrance Ramp Metering (2 GP Lanes + 1 HOV Preferential Lane)



Chapters 600 – 680 Pavement Engineering

Chapter 600 – General Aspects

Topic 601 – Introduction

Pavement engineering involves the determination of the type and thickness of pavement surface course, base, and subbase layers that in combination are cost effective and structurally adequate for the projected traffic loading, service life, and specific project conditions including climate. 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
- Results of research and observations 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 with 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 standards, procedures, and requirements found in this manual are best practices. They should not preclude engineering judgment based on experience, and knowledge of the local conditions, including drainage and continuity with existing pavement structures, when developing pavement structures for individual projects.

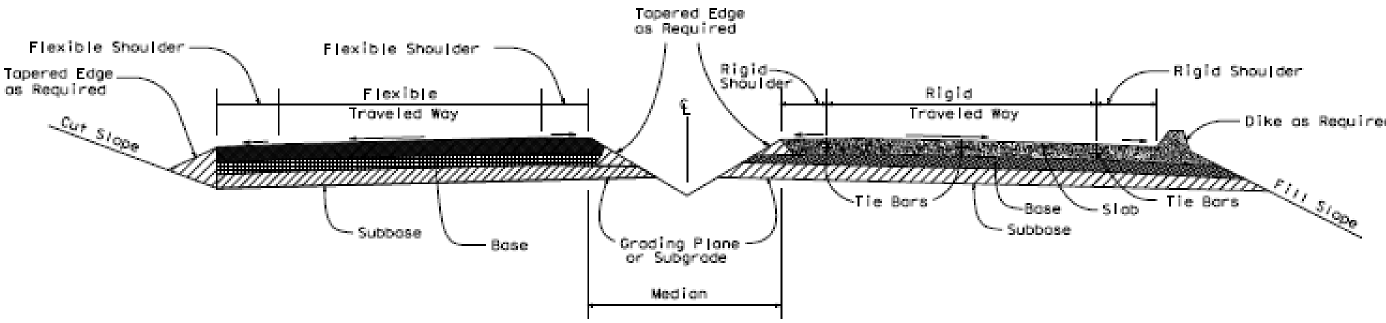
Topic 602 – Pavement Structure Layers

Index 602.1 – Description

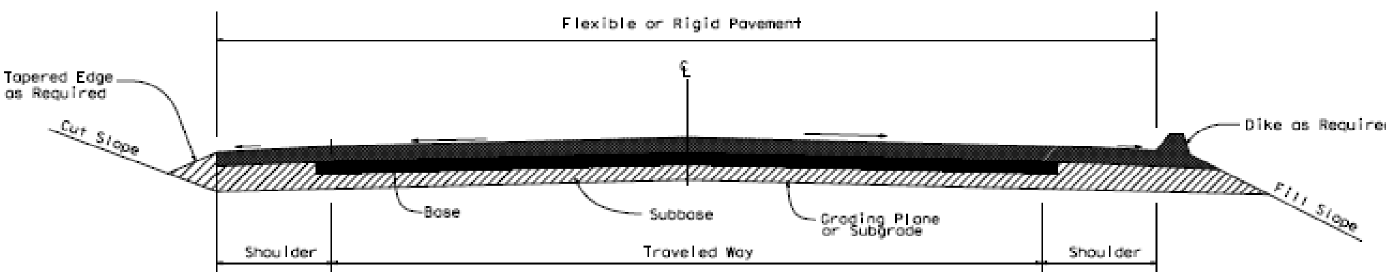
Pavement structures are comprised of one or more layers of select materials placed above the subgrade. The basic pavement layers of the roadway are shown in Figure 602.1 and discussed below.

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, is illustrated and discussed in Chapter 650.

(3) Only flexible and rigid pavements shown. Composite pavements are typically 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.

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

(2) *Subbase.* It is the unbound or treated aggregate or granular material 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, provide mass to reduce subgrade expansion, 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 high quality subgrade 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.* It is the select, processed, and/or treated aggregate material that is placed immediately below the surface course. The base may be composed of existing pavement layers that have been recycled in place. It provides additional load distribution and contributes to drainage and frost resistance. The 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 the base is typically a higher quality material than that used in the subbase. Further discussion on base materials and concepts can be found in Chapter 660.

(4) *Surface Course.* It represents one or more layers of the pavement structure engineered to accommodate and distribute traffic loads, provide skid resistance, minimize damaging 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 be composed of multiple layers of different materials. Pavements are generally classified based on the type of surface course, as follows:

   (a) Flexible Pavements. These are pavements in which the surface course, is an asphalt-bound structural layer underlain with a non-rigid base. Each layer in a flexible pavement is designed to transmit and distribute traffic loads to a level at which the next layer below can carry them adequately over the design life. The highest quality layer is the surface course, which typically consists of one or more layers of asphalt concrete and may or may not incorporate underlying layers of base and/or subbase. These pavements are called "flexible" because each layer controls flexure from traffic loads to protect the layers beneath it. Flexible pavements with cement stabilized base layers are called “semi-rigid” pavements because the cemented base controls a large part of resistance to flexure from traffic loads, in addition to the surface course. 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. The concrete surface can be composed of slabs with joints cut in them and no reinforcing steel, in which case it is referred to as jointed plain concrete pavement (JPCP), or a continuous layer of concrete with reinforcing steel is referred to as continuously reinforced concrete pavement (CRCP). These pavements rely on the substantially higher stiffness of the concrete slab alone to distribute the traffic loads over a relatively wide area of underlying layers and the subgrade. The base layer helps the concrete distribute loads, although most flexural resistance is in the concrete. The base and subbase courses are primarily used to support to the concrete slab, improve drainage, and minimize pumping of fine materials to the surface. Procedures for rigid pavements can be found in Chapter 620.

(c) Composite Pavements. These are pavements comprised of both flexible (asphalt concrete) and rigid (cement concrete) layers over underlying layers of stabilized or unstabilized base or subbase materials. In California, composite pavements consist mostly of existing rigid pavements that have been overlaid with hot mix asphalt (HMA), open graded friction course (OGFC), or rubberized hot mix asphalt (RHMA). Refer to 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. Non-structural wearing courses are also placed over pavements to reduce noise and improve wet weather skid resistance condition. Although non-structural wearing courses are not given a structural value in the pavement structural design procedures found in this manual, they will improve the service life of the pavement by protecting it from the effects of traffic and the environment.

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

(a) Interlayers can be used between pavement layers or within pavement layers to reinforce pavement and/or improve the resistance of HMA layers to reflective cracking. Interlayers can be a geosynthetic type or asphaltic chip seals. Refer to Chapter 630 and Chapter 660 for additional information.

(b) Bond Breakers are used to prevent bonding between two pavement layers such as rigid pavement surface course to a cement-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 concrete where multiple lifts are required.

(d) Prime Coats are used on aggregate base prior to paving of the surface course to provide a wearing surface for construction traffic (may require reapplication before placement of the surfacing), for better bonding with the layer to be placed above it, and to act as water proofing of the aggregate base during construction.

(e) Leveling Courses are used to fill and level surface irregularities and ruts before placing overlays. Hot mix asphalt is commonly used for constructing leveling courses.

(f) Working Platform is a layer of granular base, asphalt, or concrete used to support construction equipment. A working platform permits the efficient construction of the treated base and asphalt or concrete structural course.

Topic 603 – Types of Pavement Projects

603.1 New Construction

New construction is building a new facility, including new roadways, new alignments, interchanges or grade separation crossings, and new parking lots or safety roadside rest areas.

603.2 Widening

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

Additional guidance and requirements on widening existing facilities, including possible options as well as certain circumstances that may justify adding rehabilitation or pavement preservation work to widening, or deferring it, are discussed in Index 612.3.

603.3 Pavement Preservation

Pavement Preservation has two main categories or programs:

(1) Preventive Maintenance. Preventive maintenance projects are used to construct preventive treatments to preserve pavements in good condition. Pavement preservation consists of nonstructural preventive and corrective strategies to maintain existing pavement in generally good condition. These projects are typically done through the Highway Maintenance Program or by Department Maintenance forces. The District Maintenance Engineer determines which preventive treatment to apply and when in consultation with the District Pavement Engineer.

Traffic safety and other operational improvements, geometric upgrades, or widening are not included in preventative maintenance projects. Strategies and guidelines on preventive maintenance treatments currently used by the Department are discussed further in Indexes 624.1, 634.1, and 644.1.

(2) *Capital Preventive Maintenance (CAPM).* Capital Preventive Maintenance (CAPM), also called Minor Pavement Rehabilitation is a program of short-term (5 to 10 year years) repair projects on existing roadways in generally fair condition with considerable remaining service life (15 to 30 years) agreed to between the Department and the FHWA beginning in 1994. CAPM projects are more closely related to preventive maintenance projects than roadway rehabilitation projects because they are not intended to significantly increase or restore the pavement's structural capacity.

The primary purpose of the CAPM program is to repair pavement exhibiting minor distress as identified in Design Information Bulletin (DIB) 81 or under the current Flexible and Rigid Selection Criteria, sections 2.1.1 and 2.1.2, using condition survey data from the Automated Pavement Condition Survey (APCS) and decision trees in the Pavement Management System (PMS). Ride improvement and preservation of serviceability are key elements of this program. The timely application of CAPM treatments delays the need for major roadway rehabilitation, improving the cost-effectiveness of the pavement life cycle. CAPM provides flexibility to make the most effective use of all funds available in the biennial State Highway Operation and Protection Plan (SHOPP).

CAPM projects involve non-structural overlays and repairs, which do not require pavement structural design. CAPM projects include all appropriate items or work necessary to keep the pavement in good condition for a minimum of 5 years and up to 10 years. The District Maintenance Engineer is responsible for making strategy selections and design recommendations for CAPM projects. Information on CAPM strategies is found in Indices 624.2, 634.2, and 644.2. For further information and other guidance for CAPM projects, see DIB 81 or current DIB and Caltrans Project Development Procedures Manual (PDPM).

See DIB 81 or current for required work regarding accessibility for persons with disabilities as part of CAPM projects.

603.4 Roadway Rehabilitation

The primary purpose of roadway rehabilitation projects is to return roadways that exhibit major structural distress to good condition. Many of these structural distresses indicate failure of the surface course and underlying base layers. Roadway rehabilitation work is generally regarded as major, non-routine work engineered to restore service life by restoring damaged structural capacity and providing upgrades to enhance safety.. As described in the current Design Information Bulletin or DIB 79, Section 1.2, rehabilitation criteria also apply to minor projects and certain other projects in addition to roadway rehabilitation projects. Roadway rehabilitation is different from pavement preservation that simply preserves or repairs the facility to a good condition.

Roadway rehabilitation projects are divided into 2R (Resurfacing and Restoration) and 3R (Resurfacing, Restoration and Rehabilitation). Roadway rehabilitation projects should address other highway appurtenances such as pedestrian and bicyclist facilities, drainage facilities, lighting, signal controllers, and fencing that are failing, worn out or functionally obsolete. Also, unlike pavement preservation projects, geometric enhancements and operational improvements may be added to roadway rehabilitation work if such work is critical or required by FHWA standards.

Roadway rehabilitation strategies for rigid, flexible, and composite pavements are discussed in Topics 625, 635 and 645. Additional information and guidance on roadway rehabilitation, including determining whether the project fits 2R or 3R screening criteria, and other rehabilitation projects may also be found in the Design Information Bulletin, Number 79-04 or current - "Design Guidance and Standards for Roadway Rehabilitation Projects" and in the PDPM Chapter 9, Article 5 (https://design.onramp.dot.ca.gov/downloads/design/files/lap/PDPM_LAP_Manuals.pdf).

603.5 Reconstruction

Pavement reconstruction replaces the entire existing pavement structure with an equivalent or increased new pavement structure, and rebuilding of adjacent operational and roadside features. Reconstruction is typically warranted when the roadway has become functionally and structurally obsolete.

Reconstruction features typically include significant change to the horizontal or vertical alignment of the highway, and may include the addition of lanes. Although reconstruction is often done for reasons other than pavement repair, it can be done as an option to rehabilitation when the existing pavement meets the following conditions:

- It is in a substantially distressed condition with extensive damage or other problems in many or all of the pavement layers and rehabilitation strategies will not restore the pavement to a good condition;
- Existing alignments and clearances are functionally obsolete and need to be upgraded to improve safety and mobility;
- 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 portions of the roadway width whereas reconstruction is the removal and replacement of the entire roadway width. Incidental rebuilding of existing pavements for rehabilitation to conform to bridges, existing pavement, or to meet vertical clearance standards is considered rehabilitation and not reconstruction. Storm and earthquake damage repairs (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 carry traffic anticipated during construction temporarily . These types of pavements should be engineered using the pavement standards and procedures for new construction except where noted otherwise.

603.7 Stage Construction

In some cases, a pavement structure may need to be staged (constructed at different times or over multiple projects.) Stage construction for flexible pavement structures could be done by reducing the surface course thickness with provision for a future overlay to bring the pavement to full design depth. For rigid pavement stage construction, the base and subbase layers could initially be built (if the base is built with asphalt) and then overlaid later with concrete pavement.

Where staging of the pavement structure is needed, the initial stage:

- Needs to be built to meet or exceed the expected time of the initial stage before placing the final stage.
- Needs to meet or exceed what would be required for ultimate pavement structure when final layers are placed.
- Should show the future placement of the pavement on the typical sections.

Topic 604 – Roles, Resources, and Proprietary Items

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 responsible for 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 including pavement engineering, quality control, and estimates. This includes collaborating with the District Materials Engineer, District Maintenance Engineer 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.

(3) *District Materials Engineer (DME).* The DME is responsible for determining materials information used to develop pavement engineering strategies. 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 when needed for new construction, widening and rehabilitation projects; provides materials recommendations to and in continuous consultation with the PE throughout planning and design, as well as with the PE and Resident Engineer during construction. The DME also coordinates materials information with the Department functional units: Material Engineering and Testing Services (METS), Headquarters functional units, local agencies, industry, and consultants.

(4) *District Maintenance Engineer.* The District Maintenance Engineer 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, prioritizing pavement projects to meet those needs, and recommends pavement preservation strategies. The District Maintenance Engineer establishes pavement projects and reviews planning documents prepared by the PE for consistency with overall District and statewide goals for pavements.

(5) *Pavement Program (PP)*. The PP, within the Division of Maintenance (DOM) is responsible for statewide standards and guidelines for the pavement engineering process. The DOM Assistant Division Chief for Pavement Program serves as the State Pavement Engineer for the Department.

The PP Office of Concrete Pavement (OCP) and Asphalt Pavement (OAP) are responsible for maintaining pavement engineering standards, specifications, standard plans, design methodologies, design software, and practices that are used statewide. The OCP and OAP also provide technical expertise on material properties and products for pavements. The OCP and OAP work closely with the District Materials Engineers, Maintenance Engineers, and Resident Engineers to investigate ongoing field and pavement related issues.

(6) *State Pavement Engineer.* The State Pavement Engineer provides leadership and commitment to ensure safe, effective, and environmentally sensitive highway pavements that improve mobility across California. The State Pavement Engineer is responsible for conveying clear direction and priorities on pavement initiatives, policies, and standards that reflect departmental goals; and implementing pavement policies, standards, and specifications.

604.2 Mechanistic-Empirical Design

On March 10, 2005, the Department committed to developing Mechanistic-Empirical (ME) design methods to replace the previously used empirical methods. The Department uses ME design methods for the structural design of new construction, widening, and rehabilitation of flexible, rigid, and composite pavement. ME methods use models based on solid mechanics principles to model the primary responses of the pavement materials in terms of stresses and strains in response to traffic loading and climatic conditions, and empirical calibrations from field and test section observations to calculate the damage and distresses that result from the primary responses. The primary responses are determined using mechanistic models such as the multilayer elastic theory (MLET) or the finite element method (FEM). The empirical calibrations are also used to determine variabilities of pavement performance which are incorporated into design reliability calculations. Reliability calculations account for the probability that a pavement will not fail before its intended design life, and the percentage of pavement in a project that will fail at the design life.

Compared to previously used empirical methods, the main advantage of the ME design methods is that it makes updating of the design method to capture innovations in materials, construction, climate, and traffic and other changes in practice much faster and more accurate.

The Department has completed the calibration of the flexible and rigid pavement ME design methods using state-wide PMS performance data and historical materials data. The ME design methods are to be used for all new construction, widening, and rehabilitation projects. The ME design methods undergo periodic updating as new materials, structures, and construction practices are introduced, and new performance data becomes available. Updates are managed by the Headquarters Pavement Program and are communicated to all practitioners by the Pavement Program through change management documents that outline the changes, the reasons for the changes, and the expected effects on designs. The following are current applications of ME design for rigid and flexible pavements:

(a) Rigid Pavements - The design catalogs for rigid pavements (see Index 623.1) are based on AASHTOware™ Pavement ME software. The design catalogs are to be used for rigid pavement design on State owned and operated highways. Using AASHTOware™ Pavement ME software cannot independently design or refine data from these because the design catalogs consider other factors not currently addressed in the AASHTOware™ PavementME software. Contact the Office of Concrete Pavement for special designs requiring use of the Pavement ME software.

Additional information on concrete pavement design is given in Index 623.

(b) Flexible Pavements - The Caltrans ME (CalME) design program should be used for flexible pavement design on all flexible pavement projects on State owned or operated highways; except for the following types of projects which use predetermined strategies and/or designs:

- Pavement preservation,
- Roadside paving (including bikeways and pedestrian pathways), and
- Parking lots.

The HQ Pavement Program (Office of Asphalt Pavements) has adopted two design approaches for ME design of flexible pavements: projects using Performance Related Specifications (PRS) and non-PRS projects. PRS projects use performance related specifications for their asphalt concrete mixes. PRS projects require asphalt mixes to satisfy limits on performance test results such as fatigue life and rutting resistance as part of the job mix formula approval process. Non-PRS project refers to all other projects and asphalt materials are specified using standard specifications or QC/QA specifications. A PRS project uses project specific mixes as inputs in CalME and the reliability calculation only needs to account for project specific uncertainties. A Non-PRS project uses state-wide median materials values as inputs in CalME and the reliability calculation accounts for state-wide variability of the properties of materials delivered to the project. PRS projects may require additional testing to develop performance specifications appropriate for the specific project.

AC Long Life designs use specific types of materials for full depth asphalt reconstruction and crack, seat, and overlay concrete pavement, including a polymer modified surface course, a stiff intermediate layer, and a full depth reconstruction structures a “rich bottom” bottom course. They are used on very high traffic routes. AC Long Life projects can be designed and built using either PRS or non-PRS specifications.

Additional information on CalME is given in Chapter 630. Index 633 provides detailed information on flexible ME design procedures. Additional information on flexible pavement ME design procedures can be found on the "ME Designer's Corner" on the Pavement Program's intranet site (https://maintenance.onramp.dot.ca.gov/paveprogram/caltrans-me-designers-corner).

604.3 Pavement Recommendations

Recommendations for pavement strategies or structures for individual projects should be documented in writing. The project engineer uses the recommendations to determine the best pavement strategy for the project.

Recommendations should include the following information:

- Pavement climate zone or climate data used to prepare the recommendations.
- Design designation.
  - Not needed for non-structural recommendations such as pavement preservation or roadside paving work.
- Multiple alternatives to accomplish the purpose and need of the project and minimum design/performance standards are found in this manual, including life cycle cost analysis.
- Compliance with Section 42703 of the Public Resources Code on the use of Rubberized Hot Mix Asphalt (RHMA) alternatives. Asphalt rubber or crumb rubber modified binders should be included for asphalt pavements in accordance with Index 631.3.
- Summary of assumptions such as pavement design life.
- Reference to Materials Report used to prepare the report.
- Preparer's name. Include engineering stamp for pavement structure recommendations.

Pavement structure recommendations for new construction, widening, rehabilitation, and other situations where pavement structural requirements need to be met should be made by the Pavement Engineer and/or reviewed by the District Materials Engineer with input from the District Maintenance Engineer. The District Maintenance Engineer typically prepares recommendations for pavement preservation projects.

604.4 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 bid ability of the project contract plans, and assist maintenance in maintaining the facility. The standard plans were developed based on research and field experience and in consultation with industry. Standard plans for pavement should not be altered or modified without prior written approval from the Headquarters Pavement Program.

(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 complement and/or modify the Standard Specifications including descriptions, quality requirements, and measurement and payment for the project work and materials. When no Standard Specifications or Standard Special Provisions exist for new or proprietary items, the Pavement Program must review and concur with the special provisions. For further information, see the Specifications section on the Department Pavement website (*https://dot.ca.gov/programs/design/ccs-standard-plans-and-standard-specifications*).

(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 maintenance issues. Information in the Technical Guidance includes, but is not limited to, resources for assistance in decision making, rigid, flexible and composite pavement rehabilitation strategies, pavement preservation strategies, guidance for site investigations, guidance on in-place recycling, and guidelines for the use of various products and materials. Technical assistance is also available from the Pavement Program to assist with pavements that utilize new materials, methods, and products. These Technical Guidance documents are on the Department Pavement website (*https://maintenance.onramp.dot.ca.gov/paveprogram/pavement-program*).

(4) *Supplemental District Standards and Guidance.* Some Districts have developed additional written pavement standards and guidance to address local issues. Such guidance supplements the standards found in this manual, the Standard Plans, the Standard Specifications, and Standard Special Provisions. District guidance does not replace statewide standards unless the State Pavement Engineer has approved an exception. Supplemental District Guidance should be approved by the District Director or as delegated to the Deputy, Division Chief, or Office Chief. Supplemental District Guidance can be obtained by contacting the District Maintenance Engineer.

(5) *Department Pavement Website.* The Department Pavement website provides a one-stop resource for standards, guidance, reports, approved software, and other resource tools related to pavements. The Department Pavement website is https://dot.ca.gov/programs/maintenance/pavement .

(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 of terms and practices used in pavement engineering to aid design engineers in obtaining a better understanding of pavements. This document is not a standards manual or guideline. Because of copyright issues, the Pavement Interactive Guide is only available to Department employees on the Pavement intranet.

(7) *The AASHTO 1993 "Guide for Design of Pavement Structures" and the AASHTO 2020 "Mechanistic-Empirical Pavement Design Guide - A Manual of Practice."* Although not adopted by the Department, the AASHTO guides are comprehensive references that provide background that are helpful to those involved in engineering of pavement structures.

Topic 605 – Record Keeping

605.1 Documentation

One complete set of electronic documents, and a hard copy set of the pavement selection and design report, should be retained in District Project History files at the end of the design stage as well as subsequent construction changes to the pavement structure at the end of construction. The documentation must contain the following:

- Pavement design life (including both the construction year and design year)
- Ride quality data as measured by International Roughness Index (IRI)
- The Traffic Index (TI) and equivalent single axle loads (ESALs) or spectra and AADTT used to engineer each pavement structure
- All inputs used to run the pavement design software, which can be produced in the standard report from the software
- All reports generated by pavement design program
- All documents used to determine pavement design inputs
- Life-cycle cost analysis (including the data required for the life-cycle cost analysis) and other factors mentioned in Topic 619

605.2 Subsequent Revisions

Any subsequent changes in pavement structures must be documented and processed in accordance with the appropriate instructions stated above and with proper reference to the original design.

Topic 606 – Research and Special Designs

606.1 Research and Experimentation

Research and experimentation are undertaken on an ongoing basis to provide improved methods and standards, which take advantage of new technology, materials, and practices. They may involve investigations of new materials, construction methods, and/or new engineering procedures. Submittal of new ideas by Headquarters and District staff, especially those involved in the engineering, construction, maintenance, paving materials, and pavement performance, is encouraged. Research proposals should be sent to the Division of Research, Innovation and System Information (DRISI) and the Pavement Program in Headquarters for review and consideration. Suggestions for changes in pavement standards may also be submitted to the State Pavement Engineer. The Pavement Program must approve research proposals, pilot projects, and experimental construction features before undertaking such projects. District Maintenance Engineers, Pavement Engineers, and Material Engineers 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 for pavement performance. A comprehensive pavement performance monitoring plan should also be developed during the project planning stage under the lead of the HQ Pavement Program.

606.2 Special Designs

"Special" designs must be fully justified and submitted to the Headquarters Pavement Program. Special designs are defined as those designs that meet 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
- Utilize experimental products or procedures not covered in the engineering tables or methods found in this manual or accompanying technical guidance

Special designs must be submitted to the Headquarters Pavement Program either electronically or as hard copies. Hard copy submittals must be in duplicate. All submittals must include the proposed pavement structure(s) and a location strip map (project title sheet is acceptable). The letter of transmittal should include the following:

- Pavement design life, including both the construction year and design year (See Topic 612)
- All inputs used to run the pavement design software, which can be produced in the standard report from the software
- All reports generated by the pavement design program
- All documents used to determine pavement design inputs
- Life-cycle cost analysis (including the data required for the life-cycle cost analysis) and other factors mentioned in Topic 619

- The name of the engineering analysis and methods used in developing the “special” design(s)
- Justification for the “special” design(s)

The Pavement Program (either the Office of Concrete Pavement or Office of Asphalt Pavement) acts as the Headquarter’s focal point to obtain concurrence from the Pavement Program and other Headquarter’s functional units as needed before approving the special designs.

606.3 Proprietary Items

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

CHAPTER 610 – PAVEMENT ENGINEERING CONSIDERATIONS

Topic 611 – Factors In Selecting Pavement Type

Index 611.1 – Pavement Type Selection

The types of pavement generally considered for new construction, widening, reconstruction, and rehabilitation in California are rigid, flexible, and composite pavements. Rigid and flexible pavements are considered for all new and reconstructed pavements. For widening and rehabilitation projects, flexible or rigid pavements may be appropriate based on performance, maintainability, and constructability of new and/or existing pavement structure. Composite pavement consisting of a flexible layer placed over a rigid pavement has mostly been used to maintain and rehabilitate rigid pavements on State highway facilities.

Life-cycle cost analysis discussed in Topic 619 should be used as a decision-support tool when selecting optimal pavement structure type for a specific project.

611.2 Selection Criteria

Because physical conditions and other factors considered in selecting pavement type vary significantly from location to location, the Project Engineer must evaluate each project to determine the most appropriate and cost-effective pavement type to be used. The evaluation should be based on good engineering judgment, utilizing the best information available during the planning and design phases of the project with a systematic consideration of the following project specific conditions:

- Pavement design life
- Traffic considerations
- Soils characteristics
- Climate
- Existing pavement type and condition
- Existing drainage type and condition
- Availability of materials
- Recycling
- Maintainability
- Constructability
- Life-cycle cost analysis
- Life-cycle assessment

The above factors should be thoroughly investigated when selecting a pavement structure and addressed specifically in all project documents (PSSR,PSR, PR, PS&E, etc.). The final decision on pavement type should be the most economical design based on life-cycle cost analysis (see Topic 619) while accounting for the other considerations listed above. In addition, the Department is currently developing a tool based on the life-cycle assessment that can be valuable in supporting the selection of pavement type and rehabilitation strategies while assisting the Department in achieving its sustainability goal (see Topic 620).

The principal factors considered in selecting pavement structures are discussed in Topic 612 through Topic 619.

Topic 612 - Pavement Design Life

612.1 Definition

Pavement design life, also referred to as the performance period, is the period of time that a newly constructed or rehabilitated pavement is engineered to perform before reaching any of the performance thresholds in Table 622.2 for concrete pavements or those in Topic 633 for asphalt pavements. The selected pavement design life varies depending on the characteristics of the highway facility, the objective of the project, and projected traffic volume and loading. The pavement structure selected for any project should provide the minimum pavement design life that meets or exceeds the objective of the project as described in Topic 612.

612.2 New Construction and Reconstruction

The pavement design life for new construction and reconstruction projects shall be no less than 40 years. For roadside facilities such as parking lots and rest areas, 20-year pavement design life may be used. Realignments or other new roadways that fit the definition of spot improvement in Design Information Bulletin 79 or current DIB are considered rehabilitation for determining pavement design life**.**

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 widening. **The pavement design life for the mainline traveled way, ramp traveled way, and intersection widening projects shall either be: (a) the remaining pavement service life of the adjacent roadway (but not less than the project design period as defined in Index 103.2), (b) 20 years, or (c) 40 years depending on which pavement design life produces the lowest life-cycle costs.** Design the first 2 feet of new shoulder pavement structure in conjunction with the lane widening, or if the shoulder is expected to be converted to a traffic lane within the pavement design life, it should be engineered to provide the same pavement design life as the adjacent traveled way. All other widening projects including shoulder widening and roadside facilities should be engineered to either match the adjacent existing pavement structure or provide a 20-year design life, depending on the design life that produces the lowest life-cycle cost. Life-cycle cost analysis is discussed in Topic 619.

612.4 Pavement Preservation

(1) *Preventive Maintenance.* Because preventive maintenance projects involve non-structural overlays, seals, grinds, or repairs, they are not engineered to meet a minimum structural design life like other types of pavement projects. Their intended goal is to extend the service life and maintain ride quality of an existing pavement structure while it is in good condition. On average, the added service life can vary from a couple of years to over 7 years, depending on the strategy being used and the existing pavement condition. Effective timing of preventive maintenance is critical for it to be cost-effective. Preventive maintenance does not provide much value when placed on pavements with extensive cracking, or when placed too early.

(2) *Capital Preventive Maintenance.* The strategies used for CAPM projects have been engineered to extend the service life and maintain ride quality of a pavement that exhibits minor distress and/or triggered ride issues (Mean Roughness Index (MRI) greater than 170 inches per mile) by a minimum of 5 years. When properly engineered and placed on pavements that meet CAPM thresholds, CAPM strategies can last 5 to less than 20 years.

612.5 Roadway Rehabilitation

The minimum pavement design life for roadway rehabilitation projects shall be 20 years except for roadways with existing rigid pavements or with a current Annual Average Daily Traffic (AADT) of at least 12,000 vehicles, where the minimum pavement design life shall be either 20 or 40 years depending on which design life has the lowest life-cycle costs. At the discretion of the District, a 40-year pavement design life may be considered and evaluated for all projects with an AADT less than 12,000 using the Department's life-cycle cost analysis procedures.

612.6 Temporary Pavements and Detours

Temporary pavements and detours should be engineered to accommodate the anticipated traffic loading that the pavement will experience during the construction period. This period may range from a few months to several years depending on the type, size, and complexity of the project. Temporary pavement should not be designed to the same depth as the new traveled way and should not require a treated base.

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 and 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 or rehabilitated pavement should perform adequately for 10 or more years, and 5 or more years when placed on existing pavement as a part of pavement preservation.

Topic 613 – Traffic Considerations

613.1 Overview

Pavements are engineered to carry the truck traffic loads expected during the pavement design life. Truck traffic, which includes transit vehicles, trucks and truck-trailer vehicles, 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 that damage the pavement.

For use in the pavement management system, and to provide a quick single measure of the intensity of traffic, a Traffic Index (TI) can be calculated as discussed in Index 613.2.

TI is no longer used directly for flexible pavement design calculations and rigid pavement design catalog tables due to the change to Mechanistic – Empirical (ME) pavement design and rehabilitation methods. Traffic information needed for flexible ME design using CalME are the number of all axle loads in the first year of the design period, the percentages of total axle loads for each load range on each axle configuration (single, tandem, tridem, and quad) which is referred to as the axle load spectrum, and the annual linear growth rate for the number of axle loads over the design life. CalME allows the user to input a TI, CalME then uses the TI along with traffic information for the project location in the Caltrans traffic databases to determine the number of axle loads, the axle load spectrum, and the annual linear growth rate. Traffic information needed for rigid pavement ME design is the Annual Average Daily Truck Traffic (AADTT) in the first year , the percentages of each truck class in the total number of trucks, the number of axles and axles types for each truck class, the percentages of axle loads for each load range on each axle configuration (single, tandem, tridem, and quad) within each truck class, the hourly distribution of truck traffic, and the annual linear growth rate for the number of axle loads over the design life.

All of these traffic input data are included in the traffic database in the CalME flexible ME design software. They are also included in the traffic database used in the Caltrans Pavement ME traffic input tool for use with the Pavement ME software for rigid pavement design, and they are used with Pavement ME when updating the Caltrans rigid pavement design catalog. These traffic data are updated periodically, and simultaneously with updates to the Caltrans pavement management software traffic database updates. All ME design programs and the PMS program use the same traffic data from the Caltrans traffic count systems and the Weigh-In-Motion (WIM) system.

For ME design, the percentages of each axle load range on each type of axle are collectively called an Axle Load Spectrum (ALS). Axle load spectra have been found to show distinct patterns based on analysis of Weight-In-Motion data on California state highways, and to be predictable based on several simple traffic variables. The appropriate axle load spectrum is identified for each segment of the State highway network with consistent traffic based on these variables, and is applied to the number of trucks in each lane on the highway segment for either flexible or rigid pavement design using the same approach and design lane distribution factors. The ME design program selects the specific ALS after the designer identifies the project location information (County, Route, Postmile, Lane, Direction). For the rigid pavement design catalog, the designer enters the tables using the AADTT in the first year and the appropriate ALS (one of five typical spectra) for the location on the state highway system. Further details on Axle Load Spectra are given in Index 613.3.

Alternatively, if information indicates that prediction of future traffic based on current traffic information in the ME design traffic databases does not represent future traffic conditions well, traffic data can be input into CalME or the Pavement ME traffic input table. CalME can take a TI calculated externally to find the number of initial annual axle loadings (CalME), or an estimate of the annual axle loadings (all types summed together) can be directly input to the program. Manual calculation of TI is discussed in Indices 613.2. An initial AADTT can be estimated externally and input into the Pavement ME traffic input tool.

613.2 Manual Traffic Index Calculations

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

Current traffic volume or loading on State highways can be obtained from the following sources:

- Annual Average Daily Traffic (AADT) counts by axle classification,
- Weigh-In-Motion (WIM) station axle load data by axle classification, or
- Annual Average Daily Truck Traffic (AADTT) volume counts by axle classification.

Both AADT and AADTT on California State Highways are updated approximately annually by Headquarters Division of Traffic Operations and used to update ME and PMS traffic databases (*https://dot.ca.gov/programs/traffic-operations*).

Districts typically have established a unit within Traffic Operations or Planning specifically responsible for providing travel forecast information. The Project Engineer should coordinate with these units in their District early in the project development process to determine whether traffic data in the ME design databases for past traffic volumes should be used for future predictions, or whether local information indicates that a manual calculation of TI or initial annual axle loadings for CalME or AADTT for Pavement ME or the rigid design catalog should be done.

(2) *Design Year Annual Average Daily Truck Traffic (AADTT).* A traffic growth factor obtained from the traffic forecasting unit is used to project current AADTT to the design year AADTT for each axle classification. In its simplest form, a straight-line projection is used to project the current one-way AADTT data to the design year AADTT. When using the straight-line projection, the truck traffic data for each axle classification 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. When there is a difference between sources, Caltrans will determine which data and assumptions to use.

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

(3) *Determine the Projected Equivalent Single Axle Loads (ESALs).* The information obtained from traffic projections and Truck Weight Studies is used to develop 18-kip Equivalent Single Axle Load (ESAL) constants (see Table 613.3A). The ESAL constants represent the estimated total cumulative traffic loading for each of the four vehicle types by axle classification during the pavement design life. Due to the relatively low number of buses in comparison to trucks, buses are typically included in the 2-axle and 3-axle truck counts. However, for facilities with high percentage of buses such as high-occupancy vehicle (HOV) lanes and exclusive bus-only lanes, projected bus volumes need to be included in the projection used to determine ESALs. For these facilities and in response to the passing of Assembly Bill 1250 which increases axle weight of transit buses procured through a solicitation process, new ESAL constants must be used for all two-axle and three-axle buses; as shown in Table 613.3A. In a facility where a significant number of buses exists beside trucks, counts for the two- and three-axle trucks must be separated from counts for the two- and three-axle buses. These distinct counts must be used with the corresponding ESAL constants to calculate the total ESALs during design life.

The ESAL constants in Table 613.3A are used as multipliers of the projected AADTT for each truck type (and bus type) by axle classification to determine the total cumulative ESALs for all truck types during the pavement design life. The total cumulative ESALs for all truck types during the design life for the pavement are in turn used to determine the Traffic Index (TI) as described in Index 613.3(5). Both the total cumulative ESALs and the resulting TI are the same magnitude when engineering flexible, rigid, and composite pavement structures.

The current 10, 20, 30, and 40-year ESAL constants are shown in Table 613.3A. Note that the constants for each axle classification are linearly proportional to design life.

(4) *Lane Distribution Factors.* Traffic on multilane highways normally varies by lane with passenger cars, vans, pickups, and buses generally in the median and HOV lanes, and heavy trucks in the outside 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. These factors are also used in the calculation of TI based on the selected design life.

(5) *Traffic Index (TI) Calculation.* 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: The TI is rounded up to the nearest 0.5.

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

Where:

TI = Traffic Index for a given design life

ESAL = Total number of cumulative 18-kip Equivalent Single Axle Loads for all truck/bus types over the design life of the pavement structure calculated using the ESAL constants given in Table 613.3A

LDF = Lane Distribution Factor (see Table 613.3B)

In lieu of using the above formula, Table 613.3C can be used to determine the TI depending on the total ESAL calculated for the design life. In Table 613.3C, the TI is given for a range of ESAL values. The total ESAL values given in Table 613.3C are already adjusted for LDF.

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 20-year design life. For this reason, the Project Engineer should involve District Transportation Planning and/or Traffic Operations in determining whether the information in the ME traffic databases should be used, or whether a more realistic and appropriate TI or initial axle loadings (CalME) or AADTT (Pavement ME or catalog) for each project should be calculated based on future projections that are different. This should be determined 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 30 and 40-year ESAL constants in Table 613.3A.

613.3 Axle Load Spectra

Each axle load spectrum includes the following data:

- Truck class (FHWA Class 4 for buses through Class 13 for 7+ axle multi-trailer combinations)
- Axle type (single, tandem, tridem, and quad)
- Axle load range for each axle type and truck class (3 to 102 kips)
- The number of axle load applications within each axle load range by axle type and truck class
- The percentage of the total number of axle applications within each axle load range with respect to each axle type, truck class, and year of data; these are the normalized values of axle load applications for each axle type and truck class
- The distribution of truck traffic for each axle type and axle load range combination throughout the day
- The distribution of truck traffic for each axle type and axle load range combination throughout the year if there is significant monthly variation

Table 613.3A
ESAL Constants

Vehicle Type (by Axle Classification)	10-Year Constants	20-Year Constants	30-Year Constants	40-Year Constants
Two-axle trucks or buses	690	1,380	2,070	2,760
Three-axle trucks or buses	1,840	3,680	5,520	7,360
Four-axle trucks	2,940	5,880	8,820	11,760
Five or more-axle trucks	6,890	13,780	20,670	27,560
Two-axle buses(1)	1,380	2,760	4,140	5,520
Three-axle buses(1)	6,808	13,616	20,424	27,232

Table 613.3B
Lane Distribution Factors

Number of Mixed Flow Lanes in One Direction (2)	Factors to be Applied to Projected One-Way Annual Average Daily Truck Traffic (AADTT) (3)			
	Mixed Flow Lanes (6), (7)			
	Lane 1 (1)	Lane 2	Lane 3	Lane 4
One	1.0	-	-	-
Two	1.0	1.0	-	-
Three	0.2 (4), (5)	0.8	0.8	-
Four	0.2 (4), (5)	0.2 (4), (5)	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 and, a factor of 0.2 for other mixed flow through lanes, HOV lanes and other inside lanes (non-truck lanes).
(3) Projected one-way AADTT is the truck traffic volume expected to use the lane during the design life for the facility.
(4) TI for non-truck permitted lanes must not exceed 11 for 20-year pavement design life and 12 for 40-year pavement design life.
(5) If HOV or other inside lanes are designated (signage required) for truck use, they must be designed to the same standards as found in this table for the outside lanes.
(6) For lanes devoted exclusively to buses and/or trucks, use a factor of 1.0 based on projected AADTT of mixed-flow lanes for auxiliary and truck lanes, and a separate AADTT based on expected bus traffic for exclusive bus-only lanes.
(7) The lane distribution factors in this table represent minimum factors and, based on knowledge of local traffic conditions and sound engineering judgment, higher values may be used for specific locations when warranted.

Table 613.3C

Conversion of ESAL to Traffic Index

ESAL (1), (2)	TI (3)	ESAL (2), (3)	TI (3)
4,710		6,600,000	
	5.0		11.5
10,900		9,490,000	
	5.5		12.0
23,500		13,500,000	
	6.0		12.5
47,300		18,900,000	
	6.5		13.0
89,800		26,100,000	
	7.0		13.5
164,000		35,600,000	
	7.5		14.0
288,000		48,100,000	
	8.0		14.5
487,000		64,300,000	
	8.5		15.0
798,000		84,700,000	
	9.0		15.5
1,270,000		112,000,000	
	9.5		16.0
1,980,000		144,000,000	
	10.0		16.5
3,020,000		186,000,000	
	10.5		17.0
4,500,000		238,000,000	
	11.0		17.5 (4)
6,600,000		303,000,000	

NOTES:

(1) For ESALs less than 5,000 or greater than 300,000,000, use the TI equation to calculate design TI.See Index 613.3(3).

(2) ESAL totals already adjusted for LDF.

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

(4) For TI greater than 17.5, use the TI equation. See Index 613.3(5).

613.4 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 thicknesses for each individual lane would create complications for constructing the pavement. Therefore, use different lane distribution factors for pavement engineering of mainline lanes for a multilane highway with 3 or more lanes in each direction, based on thorough consideration of constructability issues discussed in Index 618.2, together with sound engineering judgment.

(b) Freeway and Expressway Lanes. Design traffic for all new freeway and expressway lanes, including widening and auxiliary lanes, must be the greater of either the calculated value, the generated TI of 12.0 for a 40-year pavement design life (flexible surface designed using CalME) or for an AADTT of 859 for a 40-year design life (rigid surface designed from the rigid design catalog or Pavement ME). For roadway rehabilitation projects, use the design traffic in the ME design software, or the calculated TI where called for by better information than that based on past and current traffic in the ME databases.

(c) Ramps and Connectors.

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

2. Ramps to Weigh Stations. Pavement structures for ramps to weigh stations should be engineered using the mainline outside lane traffic data which has a 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 20-year, and 40-year TI or AADTT values given in Table 613.4A for light, medium, and heavy truck traffic ramp classifications, respectively. Design life TI or AADTT should be the greater of the calculated TI or AADTT, or the TI or AADTT values in Table 613.4A. Ramp TI or AADTT should never exceed mainline TI or AADTT.

The three ramp classifications are defined as follows:

- Light Traffic Ramps - Ramps serving undeveloped or residential suburban 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 613.4A
Traffic Index (TI) Values for Ramps and Connectors

(When reliable traffic forecasts are not available)

Ramp Truck Traffic Classification	Minimum Traffic Index (TI) for CalME		Minimum First Year AADTT(2) for Rigid Design Catalog or Pavement ME	
	20-Yr Design Life	40-Yr Design Life (1)	20-Yr Design Life	40-Yr Design Life (1)
Light	8.0	9.0	29	31
Medium	10.0	11.0	186	167
Heavy	12.0	14.0	859	1,272

NOTE:

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

(2)Based on 3% annual linear growth rate

(2) Shoulders:

(a) Purpose and Objectives.

Shoulder pavement structures must be designed and constructed to assure that the following performance objectives are met:

- Be safely and economically maintained.
- Enhance the performance of adjacent travel lanes.
- Be structurally adequate to handle maintenance and emergency vehicles and to serve as emergency parking.
- Accommodate pedestrians and bicyclists as necessary.
- Provide versatility in using the shoulders as temporary detours for construction or maintenance activities in the future.
- Make it easier and more cost-effective to convert into a traffic lane as part of a future widening.
- Simplify the Contractor’s operation which leads to reduced working days and lower unit prices.
- Current and future drainage requirements are not compromised.

- Shoulders do not need to be designed to traffic lane standards to meet these objectives. To achieve these performance objectives, the following design standards apply for shoulders on the State highway.

(b) New Construction and Reconstruction.

New or reconstructed shoulders shall be designed to match the traffic data of the adjacent traffic lane when any of the following conditions apply:

- **The shoulder width is less than 5 feet.**
- **The median width is 14 feet or less**. See Index 305.5 for further paved median guidance.
- **On roads with less than two lanes in the direction of travel where there is a sustained (greater than 1 mile in length) grade of over 4 percent without a truck climbing lane.**
- **The shoulders are adjacent to exclusive truck or bus only lanes, or weigh station ramps.** This standard does not apply to mixed use (automobile plus bus) lanes, including high-occupancy vehicle (HOV) and toll (HOT) lanes.

The shoulder may also be engineered to match the design traffic of the adjacent traffic lane provided that:

- There is an identified plan (such as Regional Transportation Plan, Metropolitan Transportation Plan, Interregional Improvement Plan) to convert a shoulder into a traffic lane within the next 20 years.
- The shoulder width and cross slope are designed to meet the geometric standards of a traveled lane following the following the guidance in Topic 301 and 302.
- Agreement is obtained by the Program Fund Manager or Agency funding the project.

When the above conditions apply and the shoulder and lane will both be constructed as part of the same project, the shoulder pavement structure and its drainage should match the adjacent traffic lane for ease of construction. For asphalt pavements, the thickness of the shoulder surface course layer may be tapered from the lane surface course thickness to the shoulder pavement edge thickness of no less than 0.35 foot to address different cross slope conditions (see Figure 613.4A).

For all other cases, the following design standards shall apply:

The minimum design traffic data for the shoulder shall match the adjacent traffic lane for the first 2 feet of the outside shoulder width where the outside lane and shoulder have flexible surfaces, the first 1.0 foot of the outside shoulder as a widened slab in the outside lane where the outside lane has a rigid surface, and 1.0 foot of the inside shoulder measured from the edge of traveled way regardless of surface type. See Figure 613.4B.

For the remaining width of the shoulder, the design traffic data shall:

- **Be no less than 2 percent of the projected axle loads in the first year of the adjacent traffic lane or a TI of 5 for flexible surfaced shoulders designed using CalME, whichever is greater, and no less than 2 percent of the first year AADTT**

for rigid surfaced shoulders designed using the rigid design catalog or Pavement ME.

- **Not to exceed a TI of 9.0 for flexible surfaced shoulders, or a first year AADTT for rigid surfaces shoulders of 77 for 20-year designs and 31 for 40-year designs**.

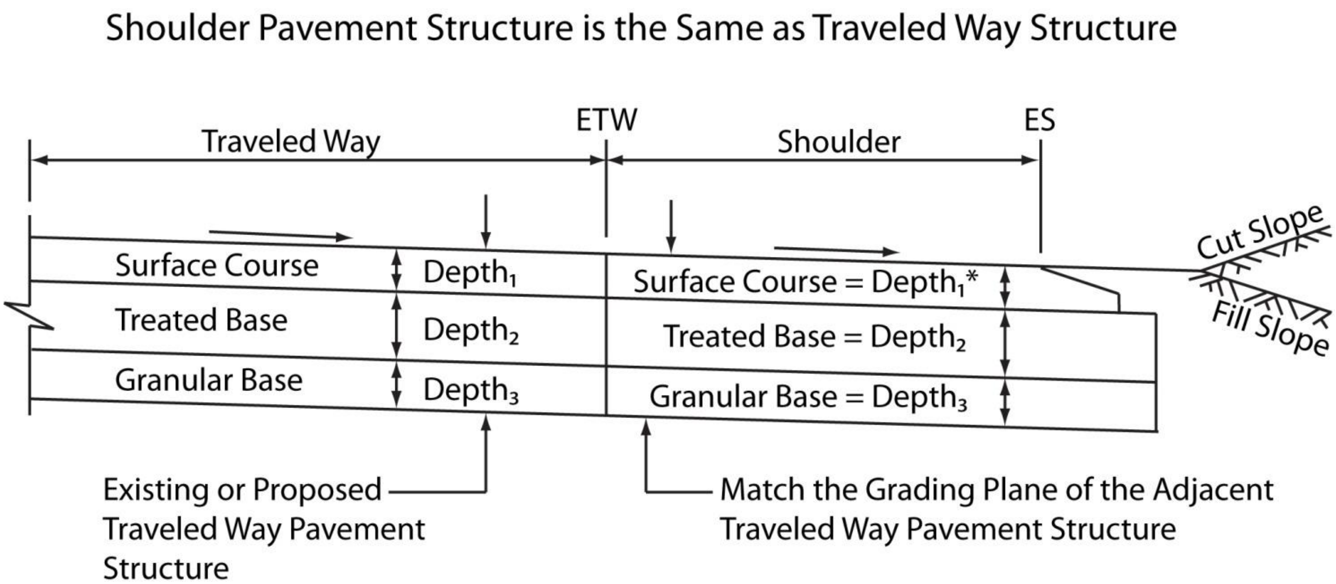
Do not use treated bases of any kind in the pavement, except when a treated permeable base is needed to provide continuity with existing treated permeable bases in the adjacent existing lane.

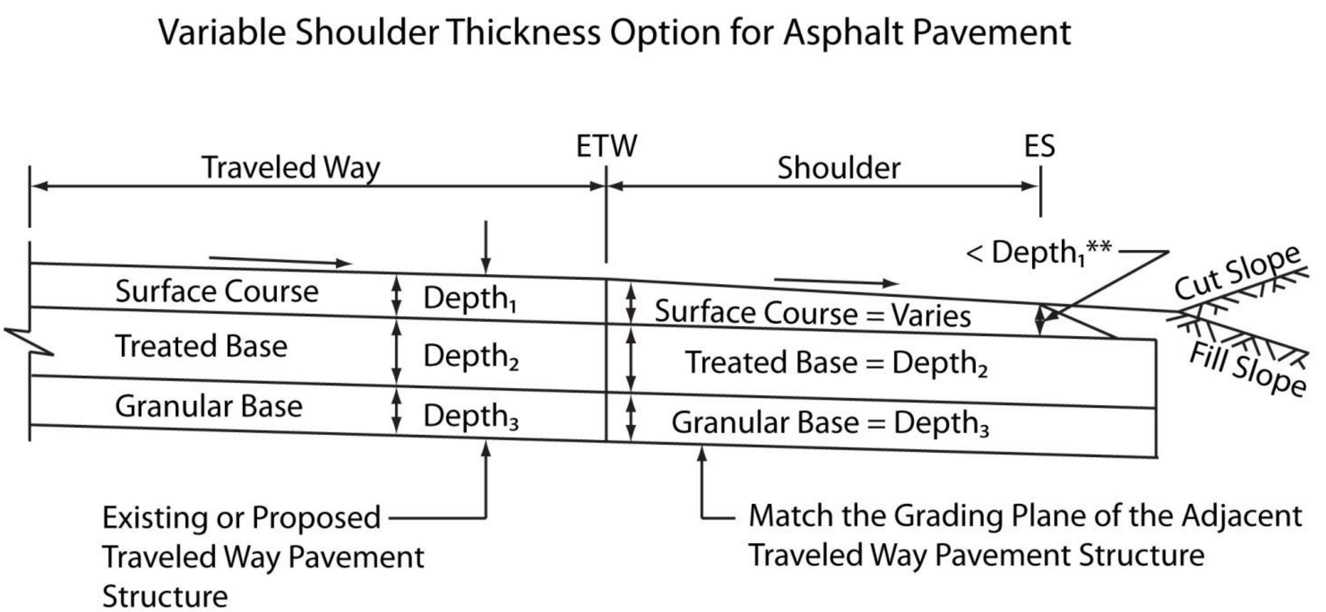
The total depth of the shoulder pavement structure (depth from the surface to the subgrade) shall match the pavement structure grading plane of the adjacent traffic lane.

Matching the total grading plane of the shoulder pavement structure to that of the adjacent traffic lane can be accomplished by increasing the depth of the aggregate base and/or subbase as needed (see Figure 613.4B). This will provide a path for water in the pavement structure to drain away from the lane and into the shoulder. It can also provide a more cost effective means to upgrade the shoulder to a traffic lane in the future. Although using a thinner overall shoulder pavement structure than the traveled way requires less material and may appear to reduce construction costs, the added costs of time and labor to the Contractor to build the step between the traveled way and shoulder can offset any perceived savings from reduced materials.

Figure 613.4A

Shoulder Design for Design Traffic or TI Equal to Adjacent Lane Design Traffic or TI





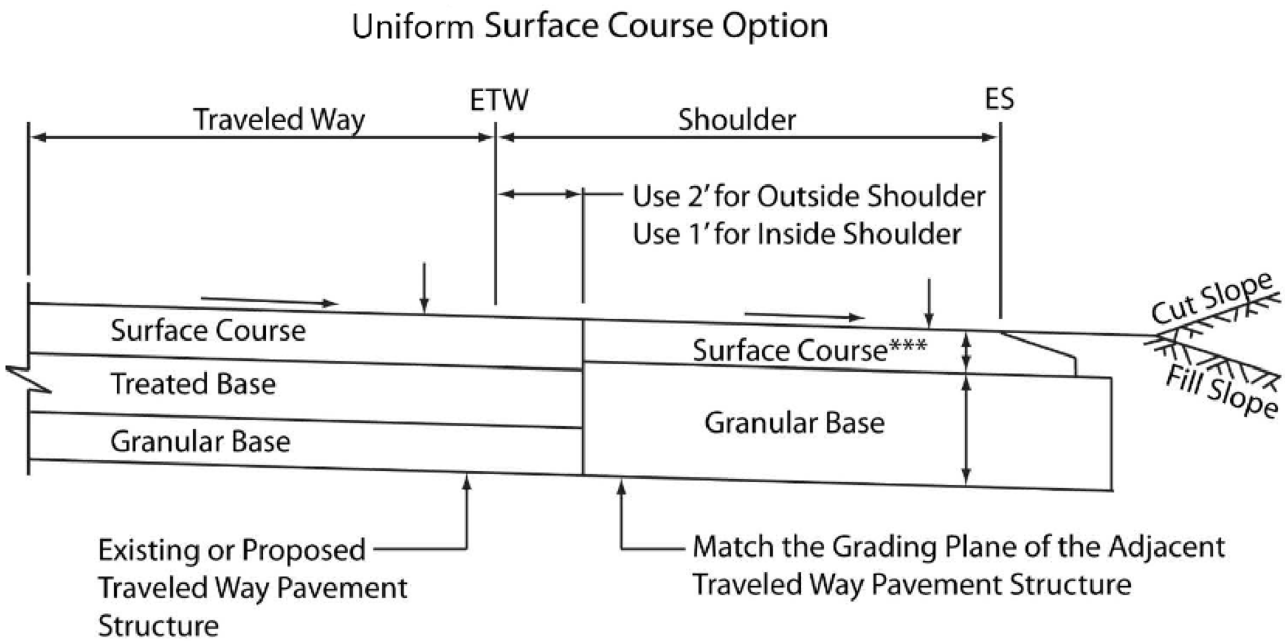
NOTES:

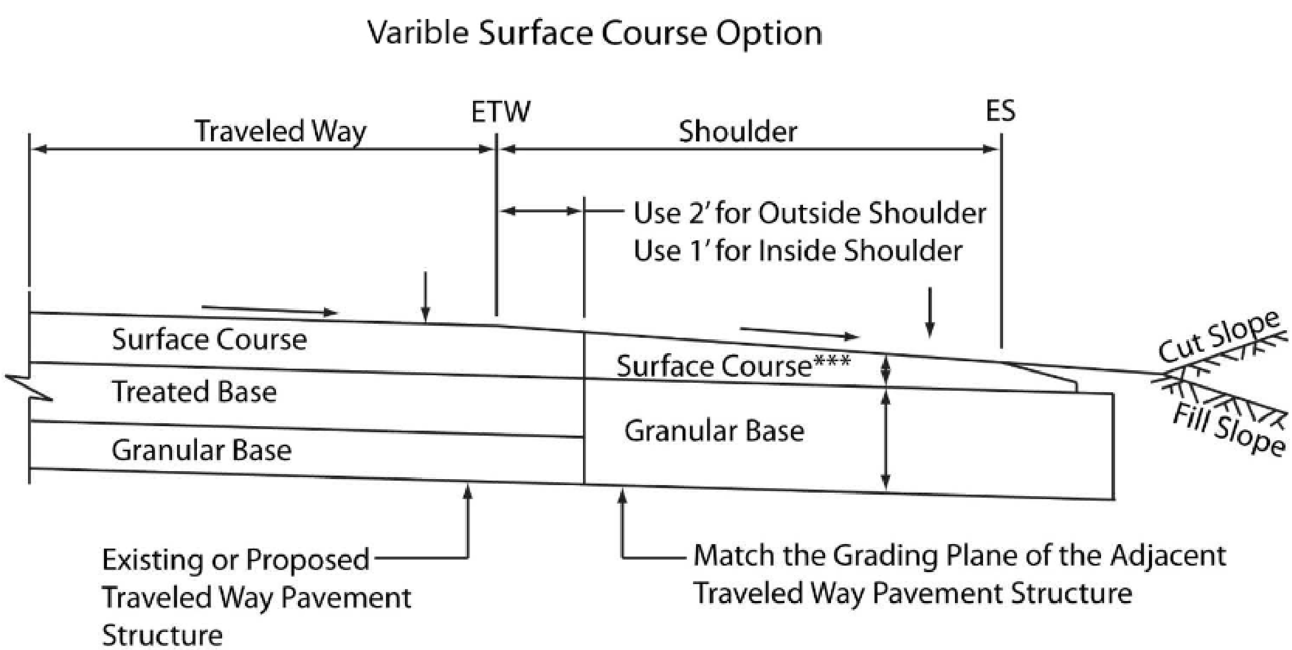
* Applies to concrete and asphalt pavements.

** For asphalt pavement, minimum thickness of surface course ≥ 0.35’.

Figure 613.4B

Shoulder Design for Design Traffic or TI Less Than Adjacent Lane Design Traffic or TI





NOTES:

*** For rigid pavement, the minimum thickness of surface course depends on climate region and shoulder type (Table 626.2). For flexible pavement, minimum thickness of surface course is 0.35.

For asphalt shoulders, the thickness of the asphalt layer (not including nonstructural wearing surface) should not be less than 0.35 foot.

For concrete shoulders, see Index 626.2 and Table 626.2 for recommended thicknesses.

An alternate shoulder design is to taper the surface course from the surface course thickness of the adjacent traffic lane to no less than 0.60 foot (0.75 foot in High Mountain and High Desert climate regions) for concrete and 0.35 foot for asphalt at the edge of shoulder (see Figure 613.4B).

Bases and subbases for new or reconstructed shoulders should extend at least 1 foot from beyond the edge of shoulder as shown in Figures 613.5A and 613.5B.

(c) Widening. Existing shoulders do not need to be replaced or upgraded to new construction or reconstruction standards as part of a shoulder widening project unless the following conditions exist:

- Adding or widening lanes will require removal of all or a portion of the existing shoulder.
- The existing shoulder of 5 feet or less in width is being widened and the existing shoulder does not meet the current standards for new construction or reconstruction. For shoulders wider than 5 feet, the District and Program Fund Manager/Agency determines whether to reconstruct the entire shoulder to new construction or reconstruction standards, or match the pavement structure of the existing shoulder.
- There is an identified plan that the widened shoulder will be converted or replaced with a traffic lane within 20 years.
- The widened shoulder will be used as a temporary detour as discussed in Index 613.4(2)(f).

For all other cases, widening of the existing shoulder should match the pavement structure of the existing shoulder. For shoulders left in place, repair any existing distresses prior to overlaying.

(d) Pavement Preservation.

Shoulder preservation should be done in conjunction with work on the adjacent traffic lanes to assure that the shoulder pavement structure will meet the performance requirements stated in Index 613.4(2)(a). Shoulders can be preserved by:

- Sealing cracks greater than ¼ inch in width,
- Grinding out rolled up sections next to concrete pavement,
- Fog or slurry sealing asphalt surfaces,
- Limited digouts of failed locations.

For CAPM projects, the following additional strategies can be considered if warranted:

- Milling and replacing 0.15 foot of oxidized and cracked surfaces can also be considered either prior to an overlay or as a stand-alone action.

- Grinding of concrete shoulders if the adjacent traffic lane is being ground.

Shoulder preservation strategies should be identified and discussed with District Maintenance and the Headquarters Pavement Reviewer during the scoping phase of the project or whenever a change in strategy is proposed.

(e) Roadway Rehabilitation.

The goal of roadway rehabilitation projects is to maintain existing shoulders wherever possible. The design truck traffic is not a consideration in choosing the shoulder rehabilitation strategy unless it has been determined that the shoulder needs to be replaced for one of the following reasons:

- The shoulder will be used to temporarily detour traffic during construction and the existing shoulder does not provide adequate structure to handle the expected loads.
- The adjacent lane is being replaced as part of the project. In this situation, if the shoulder is wider than 5 feet, replace only two feet of the outside shoulder (1.0 foot of inside shoulder) adjacent to the traffic lane. For shoulders 5 feet wide or less, replace the entire shoulder.
- The existing shoulder exhibits extensive distress and/or settlement and it is agreed to by the Headquarters Pavement Reviewer that replacement is the only viable option.

For replacements other than temporary traffic detours, use the standards for new construction and reconstruction in Index 613.4(2)(b). For temporary traffic detours, see Index 613.4(2)(f) for further discussion.

Regardless of whether or not the design truck traffic is considered, shoulder rehabilitation repairs of the existing shoulder are often necessary and should be done in conjunction with work on the adjacent traffic lanes to assure that the shoulder pavement will meet the performance requirements stated in Index 613.4(2)(a).

Existing asphalt shoulders can typically be maintained as part of a rehabilitation project by milling and replacing 0.15 feet of asphalt surface plus digouts of failed areas to remove oxidized layers. This can be done either prior to an overlay or to maintain the existing surface. Where the existing shoulders have little to no cracking and are older than 3 years from the last treatment, a fog seal or slurry seal with digouts is all that is needed.

Existing concrete shoulders typically only require sealing any unsealed cracks ½ inch or wider or replacing the joint seals. Shoulders should be sealed if the adjacent traffic lanes are sealed. If shoulders are spalled, the spalls should be repaired and any shattered slabs replaced. Grinding should not be done, even if the shoulder is faulted or curled unless the adjacent traffic lane is also being ground.

Shoulder rehabilitation strategies should be identified and discussed with District Maintenance and the Headquarters Pavement Reviewer during the scoping phase of the project or whenever a change in strategy is proposed.

(f) Temporary Detours.

When existing shoulders are used to stage traffic during construction, the existing shoulder pavement structure should be checked for structural adequacy. If the existing shoulder is not structurally adequate or if it is a new shoulder, calculate the design traffic or TI based on the actual truck traffic expected to be encountered during construction. Design the shoulder based on the requirements for new or reconstructed shoulders in Index 613.4(2)(b) except in this case the TI may exceed 9. Do not use treated bases for temporary detours. For existing shoulders, remove the surface course layer and replace with a new surface course sufficiently thick enough to support temporary traffic loads.

(g) Conversion to Lane.

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

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

If the existing shoulder is determined to be structurally inadequate for the proposed pavement design life, then the shoulder should be upgraded or replaced in accordance with the standards for new construction and reconstruction discussed in Index 613.4(2)(b).

(h) Other.

- Tracking and Sweep Width Lines.

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

- Minimizing Worker Exposure.

  Consult with District Maintenance and the Headquarters Program Advisor during the scoping phase on options for minimizing maintenance worker exposure to maintain shoulders.

- Concrete shoulders and asphalt pavement structure.

  Do not place concrete shoulders adjacent to asphalt pavement structure.

(3) Intersections.

Future ME design traffic for intersections (TI or AADTT for flexible and rigid surfaced pavements, respectively) should be determined for each approach the same way for mainline traffic. At some intersections, the level of truck/transit traffic from all approaches may add more loads on the pavement than what the mainline pavement was designed for. Separate ME design traffic calculations should be performed at intersections when any of the following criteria apply:

- Two or more State highways intersect (including ramps to/from State highways).
- Truck traffic on the local road exceeds 25 percent of the truck traffic on the State highway.
- Ramp connecting a State highway to a local road is classified as Medium or Heavy as described in Index 613.4(1)(c).

In these cases, combine the traffic of the approaches to calculate the design axle loads in the first year or TI (flexible surfaced) or AADTT (rigid surfaced) for all approaches combined. If the resulting design traffic is greater than what is calculated for the mainline, then the intersection will need to be engineered using the combined design traffic and appropriate axle load spectrum.

(4) Roundabout.

For all roundabout designs, look at the traffic projections for each turning movement of each leg of the roundabout, then, sum up the truck/transit traffic volumes using each quadrant of the roundabout. From the total truck traffic volume, generate an ME design traffic estimatefor each quadrant. Choose the quadrant with the highest design traffic to design the entire roundabout.

Special attention should be given to truck and transit traffic behavior (turning and stopping) to determine the loading patterns and to select the most appropriate materials.

The limits for engineering pavement at an intersection should include intersection approaches and departures, to the greater of the following distances:

- For signalized intersections, the limits of the approach should extend past the furthest set of signal loop detectors where trucks do the majority of their braking.
- 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.
- 100 feet.

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

For further assistance on this subject, contact either your District Materials Engineer, or Headquarters Pavement Program..

(5) Roadside Facilities.

The pavement for safety roadside rest areas, including parking lots, should meet or exceed the TI requirements found in Table 613.4B for a 20-year pavement design life for new/reconstructed or rehabilitated pavements.

Table 613.4B

Minimum TIs for Safety Roadside Rest Areas

Facility Usage	Minimum TI (20-Year) for CalME	Minimum First Year AADTT (20-Year) for Rigid Design Catalog or Pavement ME
Truck Ramps & Roads	8.0 (1)	2.9
Truck Parking Areas	6.0 (1)	2.5
Auto Roads	5.5	1.2
Auto Parking Areas	5.0	0.5

NOTE:

(1) For safety roadside rest areas next to all Interstates and those State Routes with AADTT greater than 12,000 use Table 613.4. A medium truck traffic for truck ramps, truck roads, and a minimum TI of 9.0 (CalME) or first year AADTT of 77 (rigid design catalog or Pavement ME) for 20 year designs 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. There can be considerable variation of soil types within project limits.

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 the pavement design has adequate stiffness and strength to carry the predicted traffic loads during the design life. The expansion and/or contraction potential as well as drainage conditions of the subgrade soils should also be properly addressed before a pavement is designed.

614.2 Unified Soil Classification System (USCS)

The USCS classifies soils according to their grain size distribution and plasticity. Therefore, only a sieve analysis and Atterberg limits (liquid limit, plastic limit, and plasticity index) are necessary to classify a soil in this system. Based on grain size distribution, soils are classified as either (1) coarse grained (more than 50 percent retained on the No. 200 sieve), or (2) fine grained (50 percent or more passes the No. 200 sieve). Coarse grained soils are further classified as gravels (50 percent or more of coarse fraction retained on the No. 4 sieve) or sands (50 percent or more of coarse fraction passes the No. 4 sieve); while fine grained soils are classified as inorganic or organic silts and clays and by their liquid limit (equal to or less than 50 percent, or greater than 50 percent) and plasticity index. Gravels and sands with 5 to 12 percent fines are classified by dual soil properties or symbols. 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. Testing frequency will depend on the probability of soil types changing within the project limits. Consult the Site Investigation Guide for guidance on frequency of soils sampling and testing. The soils investigation includes sampling and testing for soil classification, testing with the dynamic cone penetrometer (DCP), and also includes resilient modulus testing. DCP testing is used to evaluate variability and should be done every 500 ft or more frequently depending on visual observation of variability. Soil sampling should be done every 1,000 ft, or less frequently (minimum of one per mile) based on the variability of soil stiffness and shear strength determined from the DCP results.

614.3 Resilient Modulus, M_r

Use. The mechanistic-empirical method for flexible pavement requires the soil stiffness (as well as other unbound materials such as aggregate bases and subbases) be represented using the resilient modulus (M_r) and rigid pavement requires subgrade reaction (k). The resilient modulus is an input in the Department's CalME flexible pavement design method. The resilient modulus is determined in a laboratory using the triaxial resilient modulus test (AASHTO T 307) or can be roughly estimated from USCS classification results or dynamic cone penetrometer testing using correlation charts in the Site Investigation Guide. Where there is an existing structure, resilient modulus can be determined from the backcalculation procedure using the software program CalBack. Refer to Index 635.3 (ME Method) for additional information. For rigid pavement design the modulus of subgrade reaction (k-value) should be estimated in the Pavement ME software by inputting the AASHTO classification of the soil, which can be determined using the same information (sieve size analysis, Atterberg limits) used to do a USCS classification. The default calculation of k for rigid pavement design based on the AASHTO soil classification should always be used. Resilient modulus data from laboratory testing, backcalculated moduli, or moduli estimated by other means should not be used to estimate k.

Table 614.2

Unified Soil Classification System (from ASTM D 2487)

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

CalME Design and Analysis. In the Caltrans ME design and analysis method for flexible pavements, the resilient modulus of subgrade materials must be determined either from soil classification, DCP, back calculation, and/or M_r testing in the laboratory on soil that is typical from each design sub-section within the project. Guidance on sampling and testing to determine resilient modulus is provided in the Site Investigation Guidance document.

Resilient modulus testing in the laboratory should be performed following the AASHTO T 307 test procedure (*Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials*). Where there is more than one resilient modulus test within a sub-section, the lower value should be used from each design sub-section for input to ME design in addition to more frequent estimates of stiffness from the other methods. Sub-sections are determined from the DCP, classification and FWD data by grouping areas within the project into sub-sections with similar soils properties. Details about the selection of stiffnesses from triaxial testing, and stiffness estimates from USCS classification, DCP testing and back calculation using deflection data for design are described in the Site Investigation Guide. The resilient modulus of standard unbound engineered materials (aggregate bases, aggregate subbases) is obtained from the standard materials library available in CalME and Pavement ME. For subgrade soils, Table 614.3 lists typical resilient modulus ranges and values in pounds per square inch (psi) of subgrade soils based on their classification using the Unified Soils Classification System (see Table 614.2).

614.4 California R-Value

The R-value test used by Caltrans for flexible pavement design for more than 70 years does not give the required stiffness inputs needed for ME design and it is not the preferred method for estimation of stiffness.

Determination of R-value for subgrade is provided in California Test (CT) 301. Typical R-values used by the Department range from 5 for very soft material to 80 for unbound base material.

614.5 Expansive Soils

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

(1) Chemical treatment of expansive soil with lime or other chemical additives to reduce expansion in the presence of water. Lime is often used with highly plastic, fine-grained clay 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 subgrade strength. Selection of chemical treatment and mix design need to be determined based on laboratory testing of the soil and chemical, and not based on past experience on other projects. Swelling potential and the potential for sulfate reactions will also need to be considered during this testing. Further information on soil stabilization is available in the Soil Stabilization Guideline.

Lime stabilized soil is discussed further in Chapter 660.

Table 614.3

Typical Resilient Modulus and Poisson’s Ratios for Subgrade Soils

Subgrade Soil Classification	Resilient Modulus, M_r (ksi) (Range) Average value
CH	(3 to 13) 4
CL	(6 to 13) 9
GC	(13 to 26) 20
GW-GM	(24 to 40)
GP-GM	(19 to 37)
GW-GC	(16 to 37)
GP-GC	(16 to 35)
GM	(16 to 42) 30
GP	(18 to 35) 29
GW	(26 to 42) 38
MH	(3 to 9) 6
ML	(6 to 13) 11
SC	(6 to 16) 14
SW-SM	(13 to 27)
SP-SM	(13 to 27)
SW-SC	(11 to 25)
SP-SC	(11 to 25)
SM	(9 to 26) 21
SP	(9 to 26) 17
SW	(16 to 31) 21

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

(3) Providing a pavement structure of sufficient thickness to counteract the expansion pressure. The expansion pressure is the uplift pressure that an expansive soil layer would exert upon swelling due to saturation. The expansion pressure may be determined experimentally in the laboratory or using correlation equations that relate the pressure to a number of geotechnical properties of the soil and other site conditions such as plasticity index, density, and moisture content.

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

(5) Stabilizing the moisture content by minimizing the access of water through surface and subsurface drainage through correction of inadequate drainage is important to maintain the designed properties of the subgrade. The use of a waterproof membrane (i.e., geomembrane, asphalt saturated fabric, or rubberized asphalt membrane) in addition to drainage improvement can also be considered but is not necessarily a reliable substitute for adequate drainage.

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

Alternative (5) 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 alternative(s) is/are practical. For further assurance, more than one alternative may be selected (e.g., alternative (1) and alternative (5)).

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 Unified Soil Classification, or M_r-value of the material that is to be excavated from the cut sections of the project. If there is any excavated material that should not be used, it should be identified in the Materials Report and noted in the PS&E.

(2) Imported Borrow. Imported borrow is used in the construction of embankments/fills when sufficient quantity of quality material (M_r >12 ksi or R-value > 20 and Platicity Index (PI) <12) is not available in the cut areas of the project. When imported borrow of desired quality is not economically available or when the entire earthwork consists of borrow, the M_r-value (or R-value) specified for the borrow material becomes the design M_r-value for the pavement project. The minimum M_r-value specified for borrow material should be at least 12 ksi (R-value of 20) and PI < 12 or the M_r for the native soil, whichever is greater. Since no minimum M_r-value is required by the Standard Specifications for imported borrow, a minimum $M_r$value for the imported borrow material placed within 4 feet of the grading plane must be specified in the Materials Report and in the project plans and specifications.

(3) Compaction. Compaction is densification of the soil by mechanical means. The Standard Specifications require a relative compaction of at least 95 percent per CT 216 or 231 to be obtained between the outer edges of shoulders for the greater depth of either 0.5 foot below the grading plane or 2.5 feet below finished grade. The 95 percent relative

compaction for the depth of 0.5 foot below the grading plane or 2.5 feet below the finished grade should not be waived for the traveled way, auxiliary lanes, and ramps on State highways.

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

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

Locations where the 2.5 feet of compaction depth is waived must be shown on the typical cross sections of the project plan. If soft material below this depth is encountered, it must be removed and replaced with suitable excavated material, imported borrow or subgrade enhancement fabric. Location(s) where the Special Provisions apply should be shown on the typical cross section(s).

Topic 615 – Climate

The effects that climate will have on pavement must be considered as part of pavement engineering. Temperatures will cause pavements to expand and contract creating pressures that can cause pavements to buckle or crack. Binders in flexible pavements will also become softer at higher temperatures and more brittle at colder temperatures. Concrete temperature and drying shrinkage differences between the top and bottom of the slab cause stresses from slab curling and warping. 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 primarily based on air temperature and precipitation:

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

- Desert
- High Desert

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

In conjunction with this map, designs, standards, plans, and specifications have been and are being developed to tailor pavement standards and practices to meet each of these climatic conditions.

The standards and practices found in this manual, the Standard Plans, Standard Specifications, and Special Provisions should be considered as the minimum requirements to meet the needs of each climate region. Districts may also have additional requirements based on their local conditions. The 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

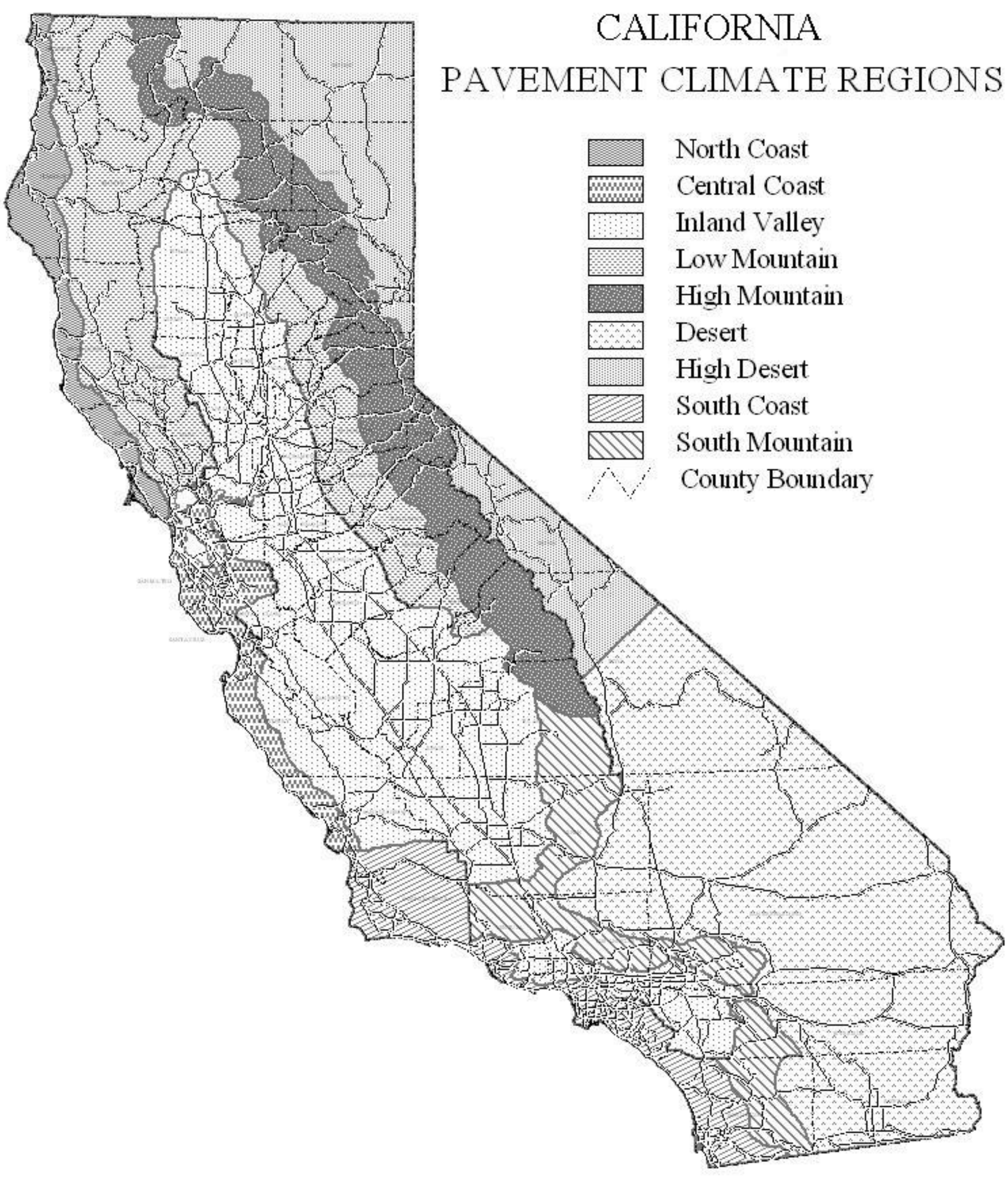
The type and condition of pavement on existing adjacent lanes or facilities should be considered when selecting new pavement structures or rehabilitation/preservation strategies. The selection process and choice made by the engineer is influenced by their experience and knowledge of existing facilities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

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 that may limit the new or rehabilitated pavement structure).

Figure 615.1

Pavement Climate Regions



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

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. Suitable materials that 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 to be surfaced with a flexible structural surface course. See the In-Place Recycling Guidelines for further information for in-place recycling. The decision to use recycled materials should be made based on a thorough evaluation of material properties, performance experience, benefit/cost analysis, and engineering judgment. In general, recycling can be broadly classified as:

- In-Place Recycling (IPR), either
  - Partial Depth Recycling (PDR) and overlay, which involves recycling of all or part of the HMA
  - Full Depth Reclamation (FDR) (also called Full-Depth Recycling) and overlay, which involves recycling of all the HMA and part of the layers beneath the HMA,
  - Cold Central Plant Recycling (CCPR), which involves recycling all or part of the HMA and can also include recycling material from beneath the HMA in a mobile plant, and
- Reconstruction, either
  - Partial-Depth Reconstruction, where all of the HMA is removed, and part of the base and subbase layers,
  - Full-Depth Reconstruction, where all of the base and subbase layers are removed down to the subgrade; the materials removed can be recycled at the site using central plant recycling (CCPR); the subgrade may be stabilized if necessary

Additional information on use of recycled pavements is available in Index 110.11 and on the Department Pavement website (https://dot.ca.gov/programs/maintenance/pavement/local-agencies/in-place-recycling-main).

617.3 Milling for Structural Changes

The Department has established a minimum remaining HMA depth after milling of 0.15 foot for recycling flexible pavement surface courses. Since existing surface course thickness will have slight variations, the recycling strategy should leave at least the bottom 0.15 foot of the existing flexible surface course in place. This is to ensure that the milling machine does not loosen base material and possibly contaminate the recycled material. If the remaining flexible pavement surface course is badly cracked, has extensive moisture damage, or otherwise will not serve as a competent material after milling to 0.15 remaining thickness, it should be fully removed. 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. Inclusion of milled RHMA, open-graded materials and preservation treatments in recycled HMA used in new HMA is allowed and does not need to be separated for inclusion in new HMA.

Topic 618 – Maintainability and Constructability

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 Constructability

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, sufficient access during closures for construction hauling of material into and out of the work zone, construction windows when the project must be completed, adequate work area, and other constructability 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 constructability during the project development process. The recommendations given by Construction should be weighed against other recommendations and requirements for the pavement. Constructability recommendations should be accommodated where practical, provide minimum performance requirements, safety, and maintainability. Some constructability items that should be addressed in the project include:

- Clearance width of paving machines to barriers and hinge points should be provided for good control of paving operation and smoothness. A minimum of 2.5 feet from limits of paving to temporary K-rail for paving machine and survey control is to be provided Access for delivery trucks and construction equipment. Delivery of consistent material is important for the paving machine to operate at a constant rate to construct smooth and long lasting pavement.
- 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, although 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, combining these layers has a negative impact on the pavement performance and will lead to untimely failure.
- Distance to material batch plant should be taken into consideration. If one is not accessible to the project site, a staging area of no less than 200 by 200 feet should be provided to produce consistent concrete or asphalt mixes and ensure proper moisture levels in aggregate mix as they are essential in creating sound and smooth pavement.
- Maximize lane closure times or utilize detours to provide consistent paving operations. Paving short sections causes more pavement tie-ins and more start-stop operations, both of which create greater potential for pavement roughness and lower durability. In lieu of short duration closures of less than 10 hours, the following traffic handling strategies should be considered for major pavement operations such as widening, rehabilitation, or reconstruction:
    - Extended weekend closures (55-hour, 48-hour, 24-hour, etc.).
    - Median widening to temporarily detour traffic.
    - Diverting some or all traffic to the opposite direction (split roadway) and using movable barriers, if needed, to maintain peak traffic flows.

- Long-term lane closures. Some roads can be at least partially closed for 2 weeks or more during light travel seasons or during entire construction.

- Order of work should be taken into consideration to ensure smooth and durable pavement. For example, diamond grinding should be done after individual slab replacement work is completed. However, for concrete pavement widening, diamond grind the adjacent existing lane prior to beginning the widening work.

Topic 619 – Pavement Life-Cycle

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 alternatives
- Different pavement design lives (20 vs. 40).

LCCA comparisons must be made between properly engineered, viable pavement structures that would be approved for construction if selected. The alternatives being evaluated should also have identical improvements. For example, comparing 20-year rehabilitation vs. 40-year rehabilitation or flexible pavement new construction vs. rigid pavement new construction, provide an identical improvement. Conversely, comparing pavement rehabilitation to new construction, or pavement overlay to pavement widening are not identical improvements.

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

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

- Pavement preservation projects (preventative maintenance and CAPM).
- 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. LCCA must be performed and documented in the PID and PA&ED phases. If a change in pavement design is done after the PA&ED, the LCCA must be updated. The Project Engineer is responsible for coordinating all aspects of LCCA and utilizing the information to ensure the most efficient use of transportation funds. Information on how to perform and document LCCA can be found in the LCCA Procedures Manual (*https://dot.ca.gov/programs/maintenance/pavement/concrete-pavement-and-pavement-foundations/life-cycle-cost-analysis*).

619.2 Life-Cycle Assessment

Life Cycle Assessment (LCA) is an approach to quantify the environmental impacts of industrial products and processes, which has been applied to pavements. The Federal Highway Administration has developed a framework and Caltrans has developed a tool called eLCAP following this framework for LCA for pavements. Using this tool, it is possible to quantify the amount of greenhouse gases (GHGs) emission (in terms of tons of carbon dioxide equivalents) and other environmental impact indicators released during the production of the various materials to be used in pavement construction, transport to the job site, and use of these materials on the project, followed by the maintenance and rehabilitation of these materials, recycling, vehicle-pavement interaction considering the effects of the pavement on fuel use, and end-of-life (i.e., a cradle-to-grave analysis). The tool will be valuable in the decision-making process regarding the selection of pavement type, materials, and rehabilitation strategies and will help the Department achieve its sustainability goals. The tool will complement the LCCA tool RealCost in the final selection of pavement materials and strategies to minimize the greenhouse gas emissions and other environmental impacts associated with pavements.

619.3 Emission Savings Analysis and Table

The Department encourages the use of more sustainable materials and construction technologies. The Emissions Table is a method of quantifying and reporting any potential savings for the Department on a project level basis. Reporting savings will also aid the Department in being accountable for its sustainability goals. When possible, after performing the LCCA and the LCA, any additional savings may be reported in an emissions savings table. The recommended savings that may be reported are as follows:

- GHG Emissions Saved Through Material, Transport, and Construction: Savings would be calculated by comparing the current strategy with conventional materials and strategies. The calculated GHG Emissions may come from the eLCAP software or the GHG Reference Document.
- Material Diverted From Landfill: The amount of recycled material diverted from landfills may be the amount of recycled material used in the project.
- Truck Trips Saved: The eLCAP program assumes an End Dump Truck carrying material to and from the job site with a 34,760-pound capacity. The number of trucks required for the project would be the quantity of material divided by the capacity, rounded up to the

next whole number. The savings may be materials that are reused, such as using RAP or In-Place Recycling.

- Cost Savings: Monetary savings may come from alternate construction practices or materials compared with conventional techniques or from LCCA against the next best strategy.

EFFICIENCY PARAMETERS	
GHG EMISSIONS SAVED THROUGH MATERIAL, TRANSPORT, AND CONSTRUCTION (MT CO_2)	
MATERIAL DIVERTED FROM LANDFILL (TONS)	
TRUCK TRIPS SAVED	
COST SAVINGS ($)	

CHAPTER 630 – FLEXIBLE PAVEMENT

Topic 631 – Types of Flexible Pavements and 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. HMA is classified by type depending on the specified aggregate gradation and mix design criteria appropriate for the project conditions. The Department uses the following types of HMA based on the aggregate gradation: (1) Dense Graded HMA, (2) Gap Graded HMA, and (3) Open Graded Friction Course. HMA types are found in the Standard Specifications and Standard Special Provisions.

631.2 Dense Graded HMA

Dense graded HMA is the most common mix used as a structural surface course. The aggregate is uniformly graded to provide for a stable and impermeable surface. The aggregate can be treated, and the asphalt binder can be modified. HMA can include recycled asphalt pavement (RAP). The Department uses one type of dense graded HMA: HMA-Type A.

631.3 Rubberized Hot Mixed Asphalt Gap Graded (RHMA-G)

Gap graded HMA is used to meet Public Resources Code section 42703 that specifies amounts of crumb rubber modifier (CRM) usage in HMA. To meet the Public Resources Code, neat asphalt binder is substituted with asphalt rubber binder containing CRM, referred to as asphalt rubber (AR) in pavement products to create rubberized HMA (RHMA). The use of gap graded aggregate creates space between the aggregate particles to accommodate larger sizes of partially digested CRM in asphalt rubber binder to make Rubberized Hot Mix Asphalt-Gap-Graded (RHMA-G). RHMA-G is used as a structural surface course. RHMA-G is commonly specified to retard reflection cracking, and resist thermal stresses created by cold temperatures and wide temperature fluctuations. RHMA-G is used as a structural surface course up to a maximum thickness of 0.20 foot. Because of maximum thickness requirements, if a thicker surface layer or overlay is called for, then an HMA layer of a predetermined thickness should be placed prior to placing the RHMA-G surface course. The minimum thickness for RHMA-G is 0.10 foot. An RHMA layer should not be placed directly on an aggregate base, except to construct a tapered edge up to a maximum of 4” wide. Do not place conventional HMA over a new RHMA-G, unless it is HMA-O. Do not place HMA or RHMA-G over an OGFC or RHMA-O.

631.4 Open Graded Friction Courses (OGFC)

OGFC, formerly known as open graded asphalt concrete (OGAC), is a non-structural wearing course placed primarily on asphalt pavement. The aggregate is open graded to provide high permeability. The primary reasons for using OGFC are the improvement of wet weather skid resistance, reduced water splash and spray, reduced nighttime wet pavement glare, and as a stormwater treatment Best Management Practice (BMP). A secondary benefit is better visibility of pavement delineation (pavement markings and pavement markers) during wet weather conditions.

Three types of non-structural OGFC are used on asphalt pavement: Hot Mix Asphalt-Open-Graded (HMA-O), Rubberized Hot Mix Asphalt-Open-Graded (RHMA-O), and Rubberized Hot Mix Asphalt-Open-Graded-High-Binder (RHMA-O-HB). HMA-O is occasionally placed on rigid pavements. The difference between RHMA-O and RHMA-G is in the gradation of the aggregate. The difference between RHMA-O and RHMA-O-HB is in the amount of binder content. The maximum thickness of HMA-O, RHMA-O or RHMA-O-HB is 0.15 foot.

Rubberized OGFC (RHMA-O or RHMA-O-HB) is recommended unless it is documented that RHMA-O and RHMA-O-HB are not suitable due to availability, cost, constructability, or environmental factors (such as a stormwater treatment BMP for National Pollutant Discharge Elimination System [NPDES] compliance). RHMA-O and RHMA-O-HB are not expected to provide a water quality benefit. The project engineer should balance the competing requirement of recycled crumb rubber goals with those for stormwater treatment and document this in the project report. Coordinate with the district pavement engineer and NPDES coordinator to determine if both goals are on target for compliance. Open-graded mixes should not be placed in areas that will not allow surface water to drain to the shoulder or median. As an example, OGFCs should not be placed in the traveled way at an elevation lower than or level with the shoulder such as when the open-graded mix is placed in a milled area of the traveled way but the shoulder has not been milled, which forms a “bathtub” section that can trap water beneath the surface of the traveled way. To prevent this effect, HMA should be placed on the milled surface (traveled way only) and an open-graded mix should then be placed over the entire cross section of the road including the traveled way and shoulders.

For additional information and applicability of OGFC in new construction and rehabilitation projects refer to the OGFC Guideline available on the Department Pavement website. Also, see the Maintenance Technical Advisory Guide (MTAG) for additional information about use of OGFC in pavement preservation. If OGFC is proposed as a stormwater treatment BMP, see the OGFC Stormwater Treatment BMP Guidance on the Design website.

631.5 Rubberized HMA (RHMA) Use

Currently, three RHMA products are used: gap-graded (RHMA-G), open-graded (RHMA-O), and open-graded-high binder (RHMA-O-HB) mixes.

The following describes situations where RHMA should not be used:

- When HMA project quantities are 1,000 tons or less or staged construction operations require less than 1,000 tons of HMA per stage, and there are no HMA production plants with full time RHMA blending plants on site.
- Where the roadway elevation is above 3,000 feet.
- When the project has a Caltrans NPDES permit requirement for stormwater treatment BMPs (only applicable for RHMA-O or RHMA-O-HB exception).

For additional information on and applicability of RHMA in new construction and rehabilitation projects refer to the Asphalt Rubber Usage Guide available on Caltrans' Intranet Office of Asphalt Pavement W*ebsite (https://maintenance.onramp.dot.ca.gov/paveprogram/office-asphalt-pavements).*

631.6 Other Types of Flexible Pavement Surface Courses

There are other types of materials used on flexible pavement surface for different purposes, such as cold mix used for patching, and other types of new materials that Caltrans evaluates. For pavement preservation and other maintenance treatments refer to the Caltrans Maintenance Manual (*https://dot.ca.gov/programs/maintenance/maintenance-manual*) and Maintenance Technical Advisory Guide (MTAG) (*https://dot.ca.gov/programs/maintenance/pavement/mtag*).

631.7 Bonded Wearing Course (BWC)

Placing a BWC consists of applying a polymer-modified asphaltic emulsion and placing the specified HMA in a single pass with an integrated paving machine. BWC is constructed using RHMA-G, RHMA-O, or HMA-O. BWC is intended to produce a longer lasting surface course than placing any of these surfaces using conventional techniques. The single pass paving and polymer modified emulsion is expected to produce a stronger bond than a conventional tack coat followed by mix placement.

631.8 Warm Mix Asphalt

Warm Mix Asphalt (WMA) is a set of technologies that permit HMA to be produced, placed, and effectively compacted at lower temperatures. WMA is particularly useful when there are long haul distances, cooler temperatures, or other conditions that produce short compaction windows. HMA may be produced for use on state highways using approved WMA technologies. The Department has an approved list of WMA additives and WMA water injection technologies that contractors may choose to use, and the standard specifications provide information for mix design and construction when the contractor chooses to use WMA. Minimum mixing temperatures, and ambient and surface temperature requirements for both the WMA additives and WMA water injection technologies are included in the standard specifications. WMA can be used for all kinds of HMA and RHMA

WMA does not change the structural design properties of HMA or RHMA-G. Therefore, all structural design methods discussed in this chapter using HMA are also applicable to WMA.

631.9 Pavement Interlayers

Pavement interlayers are used with asphalt pavement as a means to slow propagation of cracks on the surface of the existing pavement into the new flexible layer. These interlayers are not currently considered in the *CalME* mechanistic-empirical design procedure but can be used in addition to the *CalME* thickness design to provide additional reflective cracking retardation. Two types of pavement interlayers are:

- Rubberized Pavement Interlayers (RPI); also known as Rubberized Stress Absorbing Membrane Interlayer (SAMI-R); which is simply a rubberized chip seal.
- Geosynthetic Pavement Interlayer (GPI). GPI consists mainly of asphalt-saturated geotextile (also called fabric), but other geosynthetic planar products such as paving grids and paving geo-composites (grid attached to geotextile) are also used. Use of a GPI and selection of the type should consider expected potential construction difficulties during future milling and in-place recycling. Refer to the Standard Specifications for the various GPI types and installation practices.

Sound engineering judgment is required when considering the use of a pavement interlayer. The following must be considered:

- Areas that may prohibit surface water from draining out to the sides of the overlay, thus forming a “bathtub” section.
- Since a pavement interlayer can act as a moisture barrier, it should be used with caution in hot environments where it could prevent underlying moisture from evaporating.
- When placed on an existing pavement, preparation is required to prevent excess stress on the membrane. This includes sealing cracks wider than ¼ inch and repairing potholes and localized failures.

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

Pavement interlayers are also used on rigid pavements that are going to be overlaid with asphalt. An asphalt leveling course is required on the rigid pavement before placing the interlayer.

GPI is ineffective in the following applications:

- For providing added structural strength when placed in combination with new flexible pavement.
- In the reduction of thermal cracking in the new flexible pavement overlay.

When using a GPI, care must be taken to specify a product that can withstand temperatures of the asphalt placed above it, particularly for RHMA. Detailed information for selecting the appropriate type of pavement interlayer to use can be found in the MTAG on the Department Pavement website.

631.10 In-Place Recycled Flexible Pavement Layers

There are a number of approaches for in-place recycling of flexible pavements.

General terms for all types of in-place pavement recycling are:

- Hot in-place recycling (HIR): General term for all types of in-place recycling that involve heating the pavement before and during milling.
- Cold in-place recycling (CIR): General term for all types of in-place recycling that do not involve heating the pavement before and during milling. Different types of cold in-place recycling are:
  - Partial depth recycling (PDR): Rehabilitation/maintenance process where only the asphalt concrete layers are recycled (i.e., the recycler milling teeth remain in the asphalt concrete layers). Recycling depths are typically between 0.25 foot and 0.4 foot. Cold in-place recycling (CIR) is a commonly used term for this recycling strategy.
  - Full depth recycling (FDR): Rehabilitation process where the asphalt concrete as well as the underlying unbound and/or previously stabilized layers are recycled (i.e., recycler milling teeth go through the asphalt concrete layers into the underlying layer[s]). Recycling depths are typically between 0.65 foot and 1.0 foot. Projects that require milling and removal of all of the asphalt layers and potentially some of the original underlying layers in order to maintain grade should not be considered or designed as FDR projects. Instead, treatment of the remaining material with lime or cement should be considered and designed as subbase or subgrade stabilization.

- Cold central plant recycling (CCPR): Rehabilitation process where materials are milled to the required depth, transported to a nearby central plant, processed, and then transported back to the road and laid with a paver. This can be for partial- or full-depth recycling. CCPR does not have thickness limits because the recycled material can be placed in multiple lifts. Stabilization of the underlying layers after milling of the existing pavement layers can be included as part of the process to increase structural capacity without increasing grade height (i.e., an inverted pavement structure).

See the Guide for Partial and Full Depth Recycling in California for further guidance on in-place recycling (*https://dot.ca.gov/programs/maintenance/pavement/local-agencies/in-place-recycling-main*).

631.11 Bonding between Asphalt Layers

A major factor in the service life of flexible pavement is the condition of the bond between the asphalt layers. All asphalt layers need a good bond between each asphalt lift regardless of their thickness. This is achieved with tack coats, which are essential to good bonding even when asphalt lifts are being placed in a short period of time. Bonding is also important between the asphalt layer and the underlying base or recycled layer. To achieve the maximum bond between asphalt lifts and between the asphalt and underlying layers, consult the District Materials Engineer or Headquarters Office of Asphalt Pavement for options on effective bonding methods.

Topic 632 – Asphalt Binder and Mix Specifications

632.1 Binder Classification

Asphalt binders are most commonly characterized by their physical properties which directly affect asphalt pavement field performance. Binder tests and specifications in use since 2006 are based on the Superpave Performance Grade (PG) System, which considers temperature extremes that pavements in the field are expected to withstand. The PG system was developed for conventional binders. These tests and specifications are particularly designed to address three specific asphalt pavement distress types: permanent deformation (rutting), fatigue cracking, and low temperature cracking.

Effective January 1, 2013, the Department has graded modified binders, excluding asphalt rubber binders, as Performance Graded Modified (PG-M) binder. Binder modification is achieved using either crumb rubber, polymers, or both. Research is underway to extend the concepts of performance grading to asphalt rubber binders.

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 the expected project climatic conditions, traffic speed and volume, as well as desired performance reliability. 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 (64 minus 10) must meet certain performance criteria at an average seven-day maximum pavement temperature of 64ºC, at a minimum pavement temperature of –10ºC, and also at an intermediate temperature of 31ºC.

Although modified asphalt binder is more expensive than unmodified binder, 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 unmodified asphalt binders in many paving and maintenance applications.

632.2 Binder Selection

Table 632.1 shows the PG binder 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.

Binder selection based on climate region is crucial for improving the pavement resistance to temperature extremes that cause rutting and low-temperature cracking during its service life. The intermediate temperature part of the PG specification limits binder stiffness at temperatures at which most fatigue damage occurs. The intermediate specification limiting stiffness is applicable for applications of new HMA or overlays that are approximately 0.33 ft or thinner where softer binder allows the HMA to bend without excessive damage. For the same reason, polymer modified mixes which have good rutting resistance and good resistance to crack propagation, but which have low stiffness at intermediate temperatures, should not be used more than 0.25 ft below the surface of the pavement (not including open-graded mix thickness). Stiffer binder and mixes are generally preferred for thicker applications because the stiffness helps limit the amount of bending. These considerations are included in *CalME* thickness design for new pavement and rehabilitation.

Special conditions in Table 632.1 are defined as those roadways or portions of roadways that need additional attention due to conditions where slow traffic and turning movements increase the risk of rutting, such as:

- Heavy truck/bus traffic (over 10 million ESALs for 20 years.)
- Truck/bus stopping areas (parking area, rest area, loading area, etc.)

Table 632.1
Asphalt Binder Performance Grade Selection

Climate Region (6)	Binder Grade for Hot Mix Asphalt (HMA)(1), (2)				
	Dense Graded HMA		Open Graded HMA		Gap and Open Graded Rubberized Hot Mix Asphalt (RHMA)
	Typical	Special(3)	Placement Temperature		
			> 70°F	≤ 70°F	
South Coast Central Coast	PG 64-10	PG 70-10 or PG 64-28 M(4) (5)	PG 64-10	PG 64-28M	PG 64-16
Inland Valley	PG 70-10	PG 70-10 or PG 64-28 M(4) (5)	PG 70-10	PG 64-28 M	PG 64-16 or PG 70-10
North Coast	PG 64-16	PG 64-28 M(4) (5)	PG 64-16	PG 64-28 M(5)	PG 64-16
Low Mountain South Mountain	PG 64-16	PG 64-28 M(4) (5)	PG 64-16	PG 58-34 M or PG 64-28 M(5)	PG 64-16
High Mountain High Desert	PG 64-28 or PG64-28 M(4)	PG 58-34 M(4) (5)	PG 64-28	PG 58-34 M	PG 58-22
Desert	PG 70-10	PG 64-28 M(4) (5) or PG 76-22 M(4)	PG 70-10	PG 64-28 M	PG 64-16 or PG 70-10

NOTES:

(1) PG = Performance Grade

(2) M = Modified (Polymers, crumb rubber, or both)

(3) PG 76-22 M may be specified for conventional dense graded hot mix asphalt for special conditions in all climate regions when specifically requested by the District Materials Engineer.

(4) Modified binders (M) should not be used more than 0.25 ft below the top of the structural section.

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

(6) Refer to Topic 615 for determining climate region for project.

- Truck/bus stop-and-go areas (intersections, metered ramps, ramps to and from Truck Scales, etc.)
- Truck/bus climbing lanes.

The final decision, whether a roadway meets the criteria for special conditions, rests with the District. It should be noted that even though special binder grades help meet the flexible pavement requirements for high truck/bus use areas, they should not be considered as the only measure needed to meet these special conditions. The District Materials Engineer should be consulted for additional recommendations for these locations.

For more detailed information on PG binder selection, refer to the Department Pavement website.

632.3 Hot Mix Asphalt Specifications and Flexible Pavement Design

Two types of specifications are typically used for the materials design and construction of HMA, either Standard Specifications or special provision specifications for the use of Quality Control/Quality Assurance (QC/QA). Beginning in 2002 a new type of specification has been used on selected projects referred to as "performance related specifications" (PRS).

When a project uses PRS there are additional requirements for laboratory performance-related testing to develop the specifications and properties of materials to be used in the structural design using *CalME*. The materials proposed by the contractor must be demonstrated to have those same performance-related properties to be used on the project. Because the performance related properties of the HMA materials to be used in a PRS project are known when the pavement structural section is being designed, the design reliability calculations in *CalME* account for the reduced variability of the properties and increased reliability of the design. This can potentially, but not always, assist in reducing cross-sectional thicknesses. The cost benefits of reduced structural thicknesses and greater certainty regarding HMA performance and the increased costs of additional laboratory testing should be considered together when deciding whether to use PRS. PRS can also be advantageous when considering new types of HMA mixes because they require that the performance-related properties used in the structural design and included in the project specifications for fatigue, rutting, and stiffness be met or exceeded when the contractor's materials are submitted for approval. The use of PRS in a project requires approval from the Office of Asphalt Pavement.

(a) *Non-PRS HMA materials used in design.* In the CalME pavement design software, each type of HMA specified to meet either Standard Specifications or QC/QA specifications used in structural design (HMA with different PG grades including polymer modified HMA, RHMA-G, high RAP HMA) is labeled as a "non-PRS" HMA material. The properties of a mix with the state-wide median properties for stiffness, fatigue, and rutting are used in the design calculations for projects using non-PRS

HMA materials. The performance related materials properties for the state-wide median HMA materials are included in the Standard Materials Library (SML) in the CalME software based on previous testing. The default reliability levels, and other default values are used for designs with non-PRS HMA materials.

(b) *PRS HMA materials used in design*. Projects that use PRS will need to have the mixes with the specified performance related properties made available in CalME. The reliability level between projects in designed is automatically adjusted to reflect the greater certainty regarding the performance of the HMA materials by choosing “PRS” as the “Spec. Type” when running CalME.

Topic 633 - Engineering Procedures for New Construction and Reconstruction

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

Reconstruction is the replacement of the entire existing pavement structure by an equivalent or increased new pavement structure and rebuilding of adjacent operational and roadside features. The entire removed depth is replaced with a new or recycled base, or based and subbase layers, and a new HMA surface.

Refer to Topic 603 for more details about types of pavement projects.

633.1 Mechanistic-Empirical Method for New Flexible Pavement

(1) *Application.* For information on Mechanistic-Empirical design application and requirements, see Index 604.2.

(2) *Method.* The Mechanistic-Empirical (ME) method integrates the effects of traffic loading and climate on the various layers of a pavement structure at various time increments during the analyzed service life to predict pavement performances. For new construction design, a trial pavement structure comprised of layer types and thicknesses is selected and then damage and distresses are simulated with the ME method over the design life of the pavement to determine the time it takes for the pavement to reach any of the failure thresholds. The design process involves trial and error to find the optimal structure that does not fail within the service life at the designated reliability level. This requires many computations and therefore the use of a software program. To help start the design process, the CalME program provides an initial trial pavement structure estimated to approximately meet the traffic and climate needs of the project.

The ME method offers these benefits over the empirical procedure used in the past:

- Considers the performance properties of materials such as enhanced or modified HMA (e.g., PG grade specifications, RHMA, and polymer modified HMA), rather than using a generic material.
- Allows new materials to be characterized and included in the method much faster than the empirical method.
- Incorporates detailed traffic loading characteristics by using axle load spectra based on use of the Weigh-in-Motion (WIM) locations distributed across the state highway system, rather than the roughly assumed damaging effects included in calculation of ESALs and TI.
- Accounts for the effect of climate on pavement performance.
- Simulates damage and development of individual distresses that can cause the pavement to fail: fatigue cracking, reflective cracking, rutting.
- Explicitly considers design reliability using statistical consideration of the measured effects of construction quality, material properties, climate, and traffic on performance.

The ME method for designing or analyzing flexible pavement for new construction, reconstruction, requires the following:

(a) CalME Software – Caltrans has developed CalME, the mechanistic empirical software for new flexible pavement design and rehabilitation design in California. Inputs to the CalME software include:

  - Project location (district, county, route number, post mile limits), which are used by the software to determine the WIM spectrum (see Index 613) and climate region (see Topic 615).
  - Pavement design life.
  - Truck axle loadings in the first year, and linear traffic growth rate, both have default values estimated from the built-in CalME database (the same database is used in the pavement management system), or a user input Traffic Index (TI) provided by Traffic Operations for the design lane following Index 613, which CalME uses to determine the axle loadings in the first year. For verification purposes, the CalME software provides a TI calculated for the lane with maximum traffic volume at the project location based on the pavement management system traffic database. Note that these two TI values may NOT be the same unless the design lane is the truck lane.
  - Subgrade soil classification (USCS), and subgrade stiffness (resilient modulus, M_r) measured in a laboratory from field collected samples or estimated from USCS classification and dynamic cone penetrometer (DCP) testing. USCS and DCP testing results are used to determine design sub-sections when the subgrade characteristics vary significantly within the

project. These data should be determined following the Site Investigation Guide.

Trial pavement structure to be analyzed, including surface material and other base and subbase layers, each with a design thickness and material type. The non-structural surface wearing course should be excluded when conducting a CalME analysis. All materials should be selected from the built-in Standard Materials Library (SML). CalME has typical stiffnesses for each material in the SML. In addition, CalME also has typical construction variability for each material property in the SML in terms of the standard deviation of layer thickness, layer stiffness, fatigue resistance and rutting resistance.The following inputs have default values that may only be changed with permission from the Office of Asphalt Pavement:

- HMA specification type, either using the standard specification or QC/QA specification (non-PRS), or PRS. See Index 632.3 for definition of the two specification types. The Office of Asphalt Pavement should be contacted if PRS are to be used on the project. See item (b) below for details.
- Performance criteria or thresholds such as percentage cracking and total rut depth. See item (c) below for details.
- Design reliability between projects value and within project design reliability value. The default between project reliability is 95% for all projects which is handled in CalME by selecting the appropriate project type: PRS or non-PRS. The between project reliability is the same for projects where PRS specifications are used for the HMA. Between project reliability for PRS projects is handled in CalME by changing the reliability factor in the software in consultation with the Office of Asphalt Pavement based on the PRS requirements for the project. The minimum recommended within project reliability is 95% for both non-PRS and PRS projects. See item (d) below for details.

(b) Project type regarding construction specification for HMAs – The construction specification type determines the extent of testing required and how the job mix formulas (JMF) submitted by the contractor are approved. More details on the two specification types are explained as follows:

- Non-Performance Related Specifications (Non-PRS) - In this case, the engineer will use the state-wide median non-PRS HMAs available in the SML shown in CalME. No performance related testing on HMA materials is necessary.
- Performance Related Specifications (PRS) – For PRS projects, the Office of Asphalt Pavement should be consulted to determine the approach for setting the HMA performance related test properties to be used for design and for materials acceptance, if PRS method is chosen. District has an option to choose PRS depending upon available resources and laboratory facilities. The design input and the specifications can be selected based on either of the following two approaches:

(1) Examination of materials in the state-wide materials library to determine if they are regionally applicable to the project. If a set of regionally applicable materials are in the SML, the design properties and PRS can be set based on those materials.

(2) If there are no regionally applicable materials in the SML, or the project would like to consider new materials, or to develop specifications and the design based on an updated assessment of materials available in the region, the engineer develops a mix representing average local performance and submits it to the Office of Asphalt Pavement so that it is added to the SML. Note that the same mix can be used for both PRS and non-PRS project. Each HMA material (including HMA and RHMA-G) in the SML has parameters for various models needed for performance simulation. In addition, each PRS-ready HMA material has performance limits for JMF approval in the SML. Both performance model parameters and JMF limits are developed based on laboratory performance tests. See Item (f) below for more details.

- PRS is recommended to use on larger projects as determined based on total HMA quantities used on the project. PRS projects require additional costs and time for mix design and JMF approval. Cost reductions from thinner cross-sections that are greater than the increased cost of mix design and approval are more achievable on projects for higher TIs and longer design lives. Table 633.1A provides the recommended minimum criteria for use of PRS.

Table 633.1A
Selecting ME Project Type and Materials

Design Life	HMA Material	Project Specification Type
Up to 40 years	< 100,000 tons of total HMA	non-PRS
>20 years	≥ 100,000 tons of total HMA	Consider PRS(1) and/or AC Long Life(2)

NOTE:

(1) See Index 633.2(2)(b) for the descriptions of project design and testing levels. District has an option to make this decision based on available resources and laboratory facilities for testing.

(2) AC Long Life projects use a polymer modified HMA surface, a high stiffness HMA intermediate layer (often with greater than 15% RAP), and a high binder content very low air-void content Rich Bottom layer (may be omitted when placed over concrete.

(c) Performance Criteria – The performance criteria are the performance thresholds for pavement distresses considered in structural design defining the end of the pavement life. For new construction and reconstruction, CalME accounts for flexural fatigue cracking and reflective cracking when applicable in the HMA layers and total rut depth measured at the pavement surface. The corresponding thresholds must not be exceeded during the design life of the proposed pavement structure. The pavement is said to have failed as soon as one of these thresholds is reached. CalME uses the following default threshold values:

- Cracking = 5 percent of the wheelpath
- Rut depth = 0.4 inch below surface

In terms of the Mean Roughness Index (MRI), roughness, which is the mean of the International Roughness Index (IRI) in both wheelpaths, is not considered in structural design. MRI is primarily a function of the roughness achieved in construction and control of cracking and rutting through structural design. The default design rutting and cracking threshold values minimize the risk of MRI exceeding PMS criteria for keeping MRI less than 170 inches/mile due to rutting and cracking.

(d) Reliability – All final designs using CalME must use the reliability concept. In CalME, reliability includes considering of performance uncertainty from between projects variability (BPV) and within project variability (WPV). WPV is the variability of performance within a project caused by construction variabilities such as variations in layer thickness, material production, compaction, and subgrade variability. BPV is defined as all other uncertainties that affect pavement performance, which is primarily related to differences between materials delivered under the same non-PRS specification, differences between contractors, uncertainties in future traffic estimation, and variability of climate.

CalME accounts for BPV by applying calibrated built-in reliability shift factors based on state-wide variance in comparing calculated and observed performance, and accounts for WPV using Monte Carlo simulation with typical construction variability. The shift factors are lower for PRS projects compared to non-PRS projects to account for the reduced uncertainty for materials performance variability properties between projects. Reliability of a given design is calculated as the percent of projects that do not reach failure criteria within the design life in Monte Carlo simulation.

A minimum of 60 Monte Carlo simulations is required in CalME to determine the reliability of the final design within the project. When evaluating preliminary design, a lower number of simulations may be used to expedite the simulations. If the trial design is found to pass all the performance criteria, then the Engineer may gradually reduce the thickness of one or more layers and re-run the CalME analysis to find the most cost-effective structure meeting both the rutting and cracking criteria.

(e) Materials Information – All materials used in CalME must be selected from the built-in Standard Materials Library. Contact the Office of Asphalt Pavement regarding adding new HMA or other materials to the Standard Materials Library, including new HMA materials with PRS for specific projects.

In-place recycled materials, unbound materials such as aggregate base, aggregate subbase, subgrades, and chemically stabilized bases and subbases do not require any additional testing for ME design and analysis except during construction quality control and assurance (QC/QA). The selections in CalME Standard Materials Library for these materials represent state-wide median performance. There are no PRS for these materials. The statistical distributions of resilient moduli for these pavement materials are given in Chapter 660 (Table 666.1A).

(f) HMA Laboratory Testing – The ME procedure in CalME requires parameters for performance models for each HMA. HMA performance model parameters are determined from the following standard laboratory tests:

- AASHTO T 321 "Standard Method of Test for Determining the Fatigue Life of Compacted Asphalt Mixtures Subjected to Repeated Flexural Bending". This test is used to determine model parameters for HMA fatigue performance and flexural stiffness master curve.
- AASHTO T 378 "Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT)" is used to determine model parameters for HMA rutting performance.

The tests used to determine performance model parameters are subject to research and review of experience to identify better performance tests. Consult with the Office of Asphalt Pavement for the current list of performance tests before starting a PRS project.

In addition, a PRS-ready HMA material requires performance limits for JMF approval and QC/QA. The tests used for determining performance related properties listed above are also required for JMF approval. The tests required for construction QC/QA are continually updated with experience from recent projects; consult with the Office of Asphalt Pavement for current requirements.

(g) Other considerations:

- Subgrade enhancement geotextile (SEGT) on the subgrade may be considered for subgrade resilient modulus values less than 23 ksi (or equivalent R-values of 40). Refer to Chapter Topic 665 for SEGT class selection. If the subgrade is subject to chemical stabilization using an approved stabilizing agent such as lime or cement, an SEGT should not be considered.

- A minimum thickness of Class 2 Aggregate Base (AB) is required in all flexible pavement designs to provide a construction platform depending on the subgrade USCS, see Table 633.1B.
- Consult current design requirements as identified in Index 631 and Index 602.1(4) regarding HMA surface course material types.
- Additional Guidance – Additional information on the Caltrans ME methodology and on the use of CalME can be found in the CalME Software under "Instructions" or by contacting the Office of Asphalt Pavement.

Table 633.1B

Minimum Class 2 AB Thicknesses for Different Subgrade Soils

UCSC Soil Class	Default Stiffness		Min. Class 2 AB
	M_R (ksi)	Equivalent R-Value	t (ft)
GW	38.7	68	0
GP	29.9	52	0
GM	31	54	0
GC	19.9	34	0.35
SW	21.5	37	0.35
SP	18.3	31	0.35
SM	21.5	37	0.35
SC	13.8	23	0.35
ML	12.0	20	0.50
CL	9.9	16	0.50
MH*	5.9	9	0.75
CH*	4.4	6	1.00

* Consider subgrade improvement, such as imported materials, or utilizing mechanical (Geogrid) or chemical (Lime/Cement) stabilization.

633.2 Mechanistic-Empirical Designs for Reconstruction of Flexible Pavement

Reconstruction of Flexible Pavement strategy is most often used when:

- It is required to maintain existing profile grade and rehabilitation options such as Mill and Fill (defined as milling part of the existing HMA and replacing it with a new HMA overlay of the same thickness on top) do not satisfy design needs.
- The existing base and or subbase materials are failing and need to be completely replaced.
- The existing subgrade is failing.

- It is the most cost-effective strategy based on life-cycle cost analysis.

Reconstruction should be engineered using the same procedures used for new construction found in Topic 633.1 with the following exceptions:

- Subgrade stiffness may be determined by back calculation using results from deflection testing conducted on the existing pavement before it is removed.
- In addition, back calculated subgrade stiffness may also be used to determine design pavement subsection. See the Site Investigation Guide for more details.
- Some or all of the pavement materials removed can often be recycled at the site using cold central plant recycling (CCPR) instead of being hauled away from the site and replaced with new materials. The subgrade may be stabilized if it is necessary before reconstructing the structure. The base and subgrade materials selected should reflect any applicable recycling and stabilization guidance documents.

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 (MTAG)” on the Asphalt Pavement Office website (https://dot.ca.gov/programs/maintenance/pavement/asphalt-pavement) . Deflection studies are not performed for preventive maintenance projects.

634.2 Capital Preventive Maintenance (CAPM)

(1) *Warrants.* A CAPM project is warranted if any of the following criteria are met per current DIB 81-02:

- 10 % ≤ Alligator ‘B’ ≤ 30%, and MRI ≤ 170 inches per mile
- MRI >170 inches per mile
- >30% Alligator ‘B’ consider rehabilitation

(2) *Strategies.* CAPM strategies include the following options:

(a) When the MRI is less than or equal to 170 inches per mile, use 0.15 foot of RHMA-G or 0.20 foot of HMA (conventional mix or polymer modified). The preferred alternative is 0.15 foot of RHMA-G. A 0.25 foot overlay is permissible if 1-inch gradation HMA is to be used on the project. (Note: A 0.2’ RHMA-G overlay or pavement interlayer may be appropriate under certain circumstances with HQ Pavement Program Advisor or Office of Asphalt Pavement concurrence). A 0.10 foot thick OGFC (HMA-O or RHMA-O) may be added on top of the overlay thickness but may not be substituted for an RHMA-G or HMA-A layer.

For CAPM projects with MRI greater than 170 inches per mile, the standard design is to cold plane as appropriate and place a 0.25 foot asphalt overlay in two lifts consisting of 0.10 foot followed by 0.15-foot HMA or preferably 0.15 RHMA-G overlay.

(b) Partial Depth Recycling (PDR) (previously called cold in-place recycling (CIR) is an acceptable CAPM strategy for pavement where the cracking is top-down (determined from coring, see the Site Investigation Guide) with little to no base failure regardless of MRI. Recycle between 0.25 foot and 0.4 foot of the existing asphalt pavement and then cap with a 0.15 foot HMA overlay or preferably 0.15 foot RHMA-G overlay.

(c) Existing pavement may be milled or cold planed down to the depth of the overlay prior to placing the overlay for any of the above strategies. Milling or cold planning may be beneficial or even necessary to improve ride quality, maintain profile grade, maintain vertical clearance, or taper (transition) to match an existing pavement or bridge surface.

(d) Non-structural wearing courses such as open graded friction courses, chip seals, or thin overlays not to exceed 0.10 foot (0.12 foot in North Coast Climate Region) in thickness may be added to the strategies listed above.

(e) Pavement interlayers may be used in conjunction with the strategies listed above.

(f) Digouts not exceeding 20 percent of the CAPM pavement costs may be included. Digouts should be designed to provide a minimum of 20 years added service life or at least to match the existing remaining service life of the pavement.

(g) Preventive maintenance strategies may be used in lieu of the above strategies when MRI is less than 170 inches per mile and they will extend pavement service life a minimum of 10 years until the next CAPM project is warranted.

(h) CAPM strategies for OGFC, HMA-O used as a stormwater treatment BMP should replace (with the same type) in kind.

(3) Smoothness. For an asphalt pavement CAPM project with existing MRI less than 170 inches per mile at the time of PS&E, a 0.20 foot or less single lift overlay is used; which should improve ride quality to minimum smoothness requirements. An RHMA-G overlay is preferred over HMA overlay. For CAPM projects with existing MRI greater than 170 inches per mile the standard practice is to use a 0.25-foot overlay placed in two lifts. A 0.25-foot two-lift overlay strategy should restore the ride quality to minimum smoothness requirements. A 0.10 foot HMA followed by 0.15 foot RHMA-G is preferred. Or a PDR strategy may be used. For the minimum smoothness requirements, refer to the current relevant Standard Specifications, Standard Special Provisions, and nSSPs.

(4) *Site Investigation and Testing.* Site investigations following the Site Investigation Guide, as applicable, should be conducted for CAPM projects. Deflection studies are not required for CAPM projects. The roadway rehabilitation requirements for overlays (see Index 635.2(1)) and preparation of existing pavement surface (Index 635.2(8)) apply to CAPM projects. Additional details and information regarding CAPM policies and strategies can be found in Design Information Bulletin 81-02 or current DIB "Capital Preventive Maintenance Guidelines."

Topic 635 - Engineering Procedures for Flexible Pavement Rehabilitation

635.1 Rehabilitation Warrants

Locations where overall Alligator 'B' cracking exceeds the thresholds for CAPM are eligible for rehabilitation. When Alligator 'B' cracking is less than or equal to 35 percent, perform a life-cycle cost analysis (LCCA) in accordance with the requirements of Topic 619 comparing a flexible pavement rehabilitation strategy versus a CAPM strategy. Pursue a CAPM strategy when CAPM has the lowest life-cycle cost.

635.2 Mechanistic-Empirical (ME) Design Method for Rehabilitation

(1) *General.* The methods presented in this topic are for rehabilitation projects with design life between 20 and 40 years, as per Index 612.5.

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, Office of Asphalt Pavement, and other pertinent experts who are familiar with engineering, construction, materials, and maintenance of pavements in the geographical area of the project for requirements or limitations in addition to those listed in this manual.

(2) *Engineering Criteria.* Inputs to the ME design procedure for flexible pavement rehabilitation are the same as those for new construction and reconstruction designs with the following additional considerations:

- Properties of the existing layers and the subgrade that will not be removed. The material type, layer thickness and its variability, and layer modulus and its variability should be determined through site investigation following the Site Investigation Guide. Additional tests such as deflection testing using falling weight deflectometer are required. To provide reliable rehabilitation strategies, deflection studies should be done no more than 18 months prior to the start of construction. California Test Method 357 for flexible pavement deflection measurements can be obtained from the Materials Engineering and Testing Services website. If resources permit, initial deflection studies can be done at the PID and PA&ED phases for estimating purposes. However, these initial deflection studies cannot be substituted for the final deflection study.

- Potential causes for reflective cracking in asphalt overlays on flexible surfaced pavement. Pre-existing cracking in existing layers that will not be removed is a cause for reflective cracking, as is shrinkage cracking in chemically stabilized layers. The amount of pre-existing cracking is an additional input for CalME.
- Inputs to the ME design procedure for asphalt overlay of concrete pavements are similar to those for rehabilitation of flexible pavements except that the amount of pre-existing cracking is not an input. CalME models reflective cracking in the asphalt overlay from cracks and joints in the underlying concrete using typical crack and joint spacing. CalME does not model asphalt overlays on concrete pavement without crack and seat. Consult with the District Materials Engineer or the Office of Asphalt Pavement regarding use of crack and seat.
- CalME does not model asphalt overlays on continuously reinforced concrete pavements without crack and seat option. Once CRCP is cracked and seated, analyze the pavement similar to cracked and seat JPCP.
- Inputs to the ME design procedure for asphalt overlay of composite pavements are similar to those for rehabilitation of concrete pavements. CalME models reflective cracking in the new asphalt overlay from cracks and joints in the underlying concrete that have reflected up through the existing asphalt overlay using typical crack and joint spacing.

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

(3) *Data Collection.* Developing a rehabilitation strategy requires collecting background data as well as field data. See the Site Investigation Guide for more information.

(4) *In-Situ Layer Moduli Evaluation Using Back Calculation.* In-situ layer moduli are determined through a process called back calculation, using the CalBack software program. The method of back calculation uses known layer thicknesses, multilayer elastic theory, and a numerical search algorithm to determine the resilient modulus of each layer of an existing pavement structure based on deflection basin data collected from the pavement. A deflection basin describes the deflection measured on the pavement surface as a function of distance from the applied load. The results exported from CalBack can be directly imported into CalME for use in rehabilitation design. For details on

deflection data collection and back calculation, refer to the Site Investigation Guide and CalBack Manual.

For additional information on the theory of back calculation and description of CalBack procedures refer to the link "ME Designer's Corner" located on the internal Department Pavement website (https://maintenance.onramp.dot.ca.gov/paveprogram/caltrans-me-designers-corner) or by contacting the Headquarters Pavement Program Office Chief.

(5) *Engineering Criteria Mechanistic-Empirical Analysis.* The ME method analyzes a proposed rehabilitation treatment for various performance criteria as discussed in Index 633.1(2)(c). The rehabilitation design must achieve the required reliability level for the project as discussed in Index 633.1(2)(d)

(6) *Flexible Overlay/ Mill & Overlay on Existing Flexible Pavement or Composite Pavement and Flexible Overlay on Existing Rigid Pavement.* Reflective cracking should be included in CalME analysis when rehabilitation involves the overlay of cracked flexible pavement and is automatically assumed in CalME whenever a concrete layer is included in the structure. The minimum thicknesses recommended by the Department for reflective crack retardation on flexible pavements are 0.15 foot HMA or 0.10 ft RMHA-G on flexible pavements. Caltrans is conducting research to develop necessary models to allow CalME to explicitly consider the effects of geosynthetic pavement interlayer (GPI) on pavement performance. Before that research is complemented and implemented, the following are general guidelines for their use:

- A GPI can be placed under HMA/RHMA-G for additional reflective crack retardation. Ensure that the melting point of the GPI to be used on the project exceeds the RHMA-G placement temperature. Refer to the Standard Specifications for selection of GPI. The GPI should be millable and recyclable for future maintenance or rehabilitation of the project. Consult with District Maintenance and Construction for input.
- A rubberized pavement interlayer (RPI) can be placed under a rubberized or non-rubberized hot mix asphalt overlay for additional reflective crack retardation.
- Do not reduce the HMA layer thickness for the use of GPI or RPI.

Open-graded and bonded wearing courses are not included in the thickness used to address reflective cracking.

Since existing pavement thicknesses will have slight variations throughout the project length, leave at least the bottom 0.15 foot of the existing surface course intact to ensure the milling machine does not loosen the base material or contaminate the recycled mix if used. A greater thickness of existing material must be left if the site investigation indicates that the existing asphalt layers are cracked or otherwise deteriorated, or alternatively in-place recycling or complete removal of the asphalt layers and replacement with new asphalt should be considered.

(7) *Overlay Thickness to Address Ride Quality*. The Department records ride quality (also called smoothness or roughness) as part of the Annual Pavement Condition Survey. According to the FHWA, the IRI value that most motorists consider uncomfortable for flexible pavement is 170 inches per mile. When average IRI across the two wheelpaths (MRI) measurements are 170 inches per mile or greater, the engineer must address ride quality. The entire project can be divided into groups of multiple segments that will be individually analyzed for ride quality.

To improve ride quality within the motorists' comfort level, place a minimum of a 0.25 foot overlay in two lifts. Because this overlay addresses ride quality, it does not matter whether HMA or RHMA-G is used, although the latter is preferred. This could be performed using either:

- The placement of 0.10 foot HMA followed by 0.15 foot HMA, or
- The placement of 0.10 foot HMA first, followed by 0.15 foot RHMA-G.

A non-structural wearing course may be included in the ride quality thickness. Pavement interlayers do not have any effect on ride quality.

(8) *Overlay Thickness and Governing Criterion*. The overlay thickness requirements required to address structural capacity are determined using ME design. The structural adequacy requirement should be compared with the thickness required to address ride quality and the greatest thickness is selected as the overlay thickness.

(9) *In-Place Recycling (IPR)*

IPR is the collective name of a number of different rehabilitation strategies. Refer to the In-Place Recycling Guide for more information. There are two categories of IPR depending on whether the recycling is below the bottom of the existing HMA layer.

(a) *Partial Depth Recycling (PDR).* PDR is when only part of the HMA is recycled. The recycling can be done either in-place (0.25 to 0.4 foot) or at the construction site using cold central plant recycling (CCPR, any thickness as multiple lifts are possible).

(b) *Full Depth Recycling (FDR).* The FDR process pulverizes the existing asphalt and a portion of the underlying material, while simultaneously mixing with stabilizer in one pass. Refer to the In-Place Recycling guide for alternative evaluation and selection based on the results of the site investigation, which also requires consideration of the type of layer below the HMA. After pulverization and mixing, the material is compacted, graded, and overlaid with HMA and/or RHMA. FDR transforms distressed existing asphalt into stabilized base to receive a new structural surface layer. FDR can treat a variety of project conditions but is most cost effective for cracked pavement surfaces requiring digouts of 20 percent or more by paving area.

Reflective cracking is assumed in CalME analysis if full depth recycling with Portland cement (FDR-C) is used that is not specified following mix design and construction recommendations in the in-place recycling guidance. No reflective cracking from drying shrinkage cracks is assumed in CalME analysis if FDR-C is specified following those recommendations,. The pavement designer needs to put a note in the Plan to specify maximum UCS of 450 psi.

The final FDR layer thickness is determined from the initial planned recycling depth plus an additional 5 to 10 percent swell that occurs due to recycling. As an example, if the initial planned recycling depth is 0.80 foot, the final FDR depth can be 0.80 × 1.07 = 0.85 foot.

The layer modulus of the IPR layer is dependent on the material being recycled and the recycling agent/stabilizer used. Use the appropriate material for design in the CalME Standard Materials Library that matches the recycling strategy and stabilizing agent being designed for. Swelling does not need to be considered if the final grade elevation is not important. Use the thickness of FDR that will be used in the structure, with or without grade adjustment, for input to ME design.

(10) *Procedure for Concrete Overlay on Existing Flexible Pavement.* For concrete overlay on asphalt (COA) strategies (sometimes previously referred to as whitetopping), only structural adequacy needs to be addressed. To address structural adequacy, use the tables in Index 623.1 to determine the thickness of the concrete layer. The existing HMA layer may be considered as the base for the concrete overlay if it is at least 0.25 foot thick and the surface is in good condition or has been restored with an asphalt overlay prior to placing the concrete overlay. Refer to Index 620 for more details regarding design. Note that there are separate thickness tables for concrete overlays greater than 0.65 ft and for concrete overlays less than or equal to 0.65 ft. The design details for selecting the correct table are discussed in Index 620. Cold planing of the existing asphalt should be considered for several reasons: removal of surface distressed asphalt, providing an even surface that helps achieve a uniform overlay thickness, and matching geometric requirements such as bridge clearances. To provide a smooth and level grade for the concrete overlay surface layer a 0.10 foot to 0.15 foot HMA or RHMA-G leveling course may be placed on top of the existing flexible layer..

(11) *Preparation of Existing Pavement.* Existing pavement distresses should be repaired before overlaying the pavement. Cracks wider than ¼ inch should be sealed; loose pavement should be removed and replaced; and localized failures such as potholes should be repaired. Localized failure repairs should be designed to provide a minimum design life to match the pavement design life for the project, but no less than 20 years. Undesirable material such as bleeding seal coats or excessive crack sealant should be removed before paving. Existing thermoplastic traffic striping and raised pavement markers should also be removed. The Materials Report should include a reminder of these

preparations. Additional discussion of repairing existing pavement can be found on the Department Pavement website.

(12) *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 constructability, maintenance, and other requirements in Chapter 610.

635.3 Rehabilitation of Existing RHMA-G, RHMA-O and HMA-O Surfaced Flexible Pavements

The design method is based on the ME methodology (Index 635.2.) The designer may be limited in selecting the rehabilitation strategy for the pavement. In the past, RHMA-G layers tended to be more permeable than dense graded HMA if the RHMA-G layer was not subjected to QC/QA compaction specifications. More limited research since RHMA-G was included in QC/QA compaction specifications has shown that the difference in permeability between dense-graded HMA and RHMA-G is insignificant. HMA and RHMA-G should not be placed over OGFC (HMA-O or RHMA-O) because of the potential to trap water beneath the new overlay.

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 can also be considered for exit ramp termini where there is a potential for shoving or rutting. At a minimum, rigid pavement should be considered for exit ramp termini of flexible pavement ramps where a significant volume of trucks is anticipated (TI > 11.5). For the engineering of rigid pavement ramp termini, see Index 626.1(3).

636.2 Shoulders

The TI for shoulders is given in Index 613.4(2). See Index 1003.5(1) for surface quality guidance for bicyclists.

636.3 Intersections

Where intersections have "STOP" control or traffic signals, special attention is needed to the engineering of flexible pavements to minimize shoving and rutting of the surface caused by trucks braking, and early failure of detector loops. Separate pavement strategies should be developed for these intersections that may include thicker pavement structures, stiffer and/or polymer modified or other specially designed 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, consult with the District Materials Engineer or Headquarters Division of Maintenance – Pavement Program or Office of Asphalt Pavement.

636.4 Roadside Facilities

(1) *Safety Roadside Rest Areas.* Safety roadside rest area pavements should be designed using the ME method.

For truck parking areas, where the 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.* Due to the unpredictability of traffic, it is not practical to design a new park and ride facility based on traffic projections. Therefore, standard structures based on typical traffic loads have been adopted. Table 636.4 provides layer thicknesses 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. If project site-specific traffic information is available, it should be used with the standard engineering design procedures discussed in Topic 633 and Topic 635 to design new or rehabilitate existing pavement structures. The design life of 20 years may be selected for roadside facilities. Refer to Topic 612.

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

Table 636.4
Minimum Pavement Structures for Park & Ride Facilities

Resilient Modulus (California R-value) of the Subgrade Soil	Thickness of Layers	
	HMA (2) (ft)	AB (ft)
Less than 23 ksi (40) (1) (two options)	0.25	0
	0.15	0.35
Greater than or equal to 23 ksi (40) but less than 34 ksi (60)	0.15	0
Greater than or equal to 34 ksi (60)	Penetration Treatment (3)	

NOTES:
(1) Check for expansive soil and possible need for treatment per Index 614.4.
(2) Place HMA in one lift to provide for maximum density.
(3) 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 designing flexible pavements using the procedures discussed in this chapter can be found on the Department Pavement website. These programs employ the procedures and requirements for flexible pavement engineering enabling the engineer to compare numerous combinations of materials in seeking the most cost effective pavement structure.

CHAPTER 640 – COMPOSITE PAVEMENTS

Topic 641 – Types of Composite Pavement

Index 641.1 – Asphalt Over Concrete Pavement

This configuration consists of an asphalt layer over concrete surface layer (typically jointed plain concrete pavement or continuous reinforced concrete pavement).

The asphalt layer can be designed to provide structural value or to address functional goals for the pavement surface (asphalt layers over lean concrete base or cement treated base are called semi-rigid pavement, and are considered to be flexible pavements for the purposes of this manual). For new composite pavement, the primary function of the asphalt layer is to act as a thermal and moisture blanket to reduce the vertical temperature and moisture gradients within the underlying concrete layer and decrease the deformations caused by curling and warping of concrete slabs caused by those gradients.

Asphalt over concrete composite pavements are found most often where older pavements that have had asphalt overlay such as hot mix asphalt, open graded friction course, or rubberized hot mix asphalt, placed over previously built jointed plain concrete pavement (JPCP) or continuously reinforced concrete pavement (CRCP.) New or reconstructed composite pavements consisting of asphalt layer over JPCP or CRCP typically have not been built in the past on State highways. Reasons include the typical need to replace the surface of flexible pavements more frequently than the need to recondition the surface of rigid pavements and the fact that current design methods do not consider the effects of reduced thermal and moisture gradients from an asphalt overlay in concrete thickness design. Some cases in which the asphalt over concrete composite pavement option is used include:

- To match the existing pavement structure when widening;
- When adding truck lanes to an adjacent flexible pavement;
- To provide a nonstructural surface course to an existing rigid pavement that is still structurally sound but is rough or has other surface conditions needing attention.

Thin flexible layers (i.e. sacrificial wearing course) that are 0.25 foot thick or less have sometimes been placed over JPCP or CRCP to improve the ride quality or friction of the rigid layer and to reduce tire/pavement noise. Because ride quality and friction can also be improved by diamond grinding or grooving the existing concrete layer, the Engineer should perform a life-cycle cost analysis (LCCA) to determine if diamond grinding/grooving or an asphalt nonstructural overlay is the most cost effective before deciding which option to select.

641.2 Concrete Over Asphalt Pavement

This is generally not considered composite pavement. This type of pavement is discussed in Chapter 620.

Topic 642 – Engineering Criteria

642.1 Engineering Properties

The engineering properties found in Index 622.1 for rigid pavement and Topic 631 for flexible pavement apply to composite pavements and are considered in the ME design methods for flexible overlays on rigid pavement and rigid overlays on flexible pavement. Care should be taken in selecting asphalt materials specifically to slow reflective crack propagation from joints and cracks in the underlying concrete layer when using thin asphalt overlays for preservation and CAPM overlays.

642.2 Performance Factors

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

(1) *Reflective Cracking*. Joints or cracks from the underlying concrete surface layer should not reflect through the asphalt layer during the service life of the composite pavement.

(2) *Smoothness*. The asphalt layer should be engineered to provide an initial MRI meeting the requirements of construction smoothness specifications and maintain an MRI that is less than 170 inches per mile through its design life.

(3) *Bonding.* A major factor in the service life of the composite pavement is the condition of the bond between the asphalt layers, and between the asphalt and concrete layers. Flexible layers on concrete need a good bond to the concrete and between each asphalt lift regardless of their thickness, as bonding plays an important role in the service life of the overlay. To achieve the maximum bond between asphalt and concrete layers, consult the District Materials Engineer or Headquarters Office of Asphalt Pavement for options on effective bonding methods.

642.3 Overlay Limits

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

Topic 643 – Engineering Procedures for New Construction and Reconstruction

643.1 Mechanistic-Empirical Design Method

As with all new pavement decisions, LCCA should be used to determine whether the composite pavement is more cost effective over the analysis period than asphalt or concrete pavement alternatives.

Topic 644 – Engineering Procedures for Pavement Preservation

644.1 Preventive Maintenance

Preventive Maintenance is used to maintain the asphalt surface course layer or to replace thin asphalt layers (i.e., non-structural wearing courses) placed over the underlying concrete layer. Note: Thin asphalt overlays on concrete, less than 0.35 ft thick, which includes all preventive maintenance overlays, tend to have very short reflective cracking lives. If work is needed to repair the underlying concrete layer, it should be developed as a CAPM (Index 644.2) or roadway rehabilitation (Topic 645) project. Additional information on preventive maintenance of the asphalt layer of a composite pavement is the same as for the flexible pavements, which can be found in the “Maintenance Technical Advisory Guide (MTAG)” available on the Department Pavement *website (https://dot.ca.gov/programs/maintenance/pavement/asphalt-pavement).*

644.2 Capital Preventive Maintenance (CAPM)

The CAPM warrants for concrete and asphalt pavements in Index 624.2 and 634.2 apply to composite pavements. The procedures and designs for asphalt over concrete composite pavement CAPM projects are the same as those for flexible pavements (see Index 634.2) except that instead of digouts concrete slab replacement and/or base repair may be required.

The roadway rehabilitation requirements for overlays and preparation of existing pavement surface for CAPM projects are discussed in Index 645.1. Additional details and information regarding CAPM policies and strategies can be found in Index 603.3, and Design Information Bulletin 81 or current DIB “Capital Preventive Maintenance Guidelines”.

Topic 645 – Engineering Procedures for Pavement Rehabilitation

645.1 General Considerations

For additional information on rehabilitation of composite pavement with rigid surface courses refer to the Concrete Pavement Guide available on the Department Pavement website.

Asphalt overlays on existing concrete pavement (crack and seat) are designed using CalME. The following preparatory activities should be included in projects for asphalt overlay of existing concrete pavement and asphalt overlay on existing asphalt over concrete composite pavement:

(1) Ride Quality (Smoothness). When the smoothness of the existing roadway is greater than 170 inches per mile as measured by the Mean Roughness Index (MRI), the minimum thickness should be 0.25 foot of HMA or consisting of 0.10 foot HMA followed by a minimum of 0.15 foot RHMA surface course layer as applicable. A nonstructural open-graded wearing course may be placed on the top lift. Note that in some cases the existing pavement will need to be repaired to assure the roadway smoothness will remain below 170 inches per mile throughout the life of the overlay.

(2) Preparation for Placing Asphalt Layer Over Existing Concrete Pavement.

Existing concrete slabs should generally be subjected to crack and seat procedures following Section 30 of the standard specifications. Undesirable material such as excessive crack sealant should be removed before paving. Existing thermoplastic traffic striping and raised pavement markers should also be removed. Spalls in joints and cracks should be repaired. Shattered slabs and corner cracks that exhibit pumping or have become punchouts in JPCP should be replaced, and punchouts in CRCP should be repaired. A leveling course and pavement interlayer are required for structural asphalt overlays on concrete pavement. The leveling course is 0.1 ft if the TI in the design lane is less than 12 and 0.15 ft if the TI in the design lane is greater than or equal to 12. The "asphalt impregnated fabric interlayer" is placed on the interlayer. The leveling course and interlayer are not considered part of the structural design when using CalME. Truck traffic on the leveling course prior to placement of the structural layers should be minimized to limit damage to the leveling course.

Guidance on evaluation of existing concrete pavements for asphalt overlay are included in the Site Investigation Guide.

(3) Preparation for Placing Asphalt Layer Over Existing Asphalt on Concrete Composite Pavement.

Existing non-structural wearing courses should be removed and, if needed, underlying pavement repaired prior to placing a new asphalt wearing course. In general, any existing asphalt materials that have poor bonding between lifts, moisture damage, or other concerns that may harm the performance of the new asphalt overlay should be removed. Leaving less than 0.2 foot of existing structural HMA before placing the new asphalt overlay should also be evaluated for cost-effectiveness if there are concerns that it will be rough after milling. A 0.2 foot layer provides a smoother riding surface after milling because of uncertainty of the condition of the underneath concrete pavement surface when used as a riding surface during construction.

Existing pavement distresses should be repaired before overlaying the pavement. Cracks wider than 3/8 inch should be sealed or repaired. Undesirable material such as excessive crack sealant should be removed before paving. Existing thermoplastic traffic striping and raised pavement markers also should be removed. Loose asphalt wearing course should be removed and replaced, and potholes and localized failures in the underlying concrete repaired. Corner punchouts and shattered slabs in the underlying JPCP that will impact the smoothness of the new asphalt overlay should be repaired. Punchouts in underlying CRCP should be repaired before overlay for the same reason.

645.2 Mechanistic-Empirical Design Method

For information on Mechanistic-Empirical Design and requirements, see Index 604.2.

When engineering a flexible overlay over existing JPCP, follow Index 635.2 for pavement design. Identify the existing JPCP layer as cracked in the CalME analysis.

When engineering a flexible overlay over existing CRCP, follow Index 635.2 for pavement design assuming that the CRCP is JPCP. Identify the existing CRCP layer as cracked in the CalME analysis.

There are different design procedures for the design of concrete overlays on asphalt pavement. One method uses concrete overlays with a minimum 0.65 foot thickness requirement, which are engineered similar to new concrete pavement according to the standards and procedures for rigid pavements in Chapter 620. The Department is developing design methods and engineering standards for thinner concrete overlays (0.35 to 0.60 foot) on existing asphalt pavement which are also discussed in Chapter 620; contact the Office of Concrete Pavement for further information.

Chapter 660 – PAVEMENT FOUNDATIONS

Topic 661 - Engineering Considerations

Index 661.1 – Description

Pavement foundations are the layers below the surface material, typically asphalt concrete or portland cement concrete (PCC). Pavement foundations include various types of the following pavement layers:

- Base
- Subbase including stabilized subgrade soils
- Subgrade or basement soil

Depending on the type of pavement project and other design considerations, a pavement structure may include base and subbase layers, only base, or no base or subbase. The subbase generally consists of lower quality materials than the base, but better than the subgrade or basement soils. When needed, pavement foundation materials are stabilized to improve strength. Typically, there are two types of stabilization: mechanical and chemical stabilization. Stabilization should not be used to attempt to correct problems with poor drainage, as it will seldom be successful.

Mechanical stabilization by compaction to standards for subgrade soil is essential to improve the mechanical properties of the soil and its resistance to the effects of water. Structural design methods assume that base, subbase and subgrade layers will be mechanically stabilized by compaction to standards and the design must meet performance requirements. The only cases where subgrade compaction should not meet standard specifications are for pervious pavement (see Caltrans guidance for pervious pavements) or when recommended by the district materials engineers for site-specific reasons.

Additional mechanical stabilization can be obtained in appropriate applications by using geosynthetic interlayers. Interlayers are included in the pavement for specific purposes, such as filtration of fine particles, improving confinement of granular materials, and improving constructability on very wet subgrades. Specific interlayer types are used for each purpose.

The most common chemical stabilization is performed by using cement and/or lime.

661.2 Purpose

Pavement foundations provide support for the surface layer and help distribute the wheel loads to the subgrade material.

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

- Low permeability will slow the upward pumping of fine particles from the subgrade soil into the pavement layers above.

- Minimize damage caused by frost by preventing frost heave and the effects of thawing; in areas of the state where frost penetration occurs, use materials that are not frost-susceptible at depths below the surface of the pavement reached by freezing temperatures.
- Prevent the accumulation of water within or below the pavement structure by draining laterally out of the shoulder of the pavement and leave the structure (known as "daylighting").
- Provide a working platform for construction equipment to prevent rutting in the subgrade during construction and to achieve optimal densities during compaction of subsequent layers.

Topic 662 - Types of Bases and Subbase

662.1 Aggregate Base

Aggregate bases consist of a combination of sand, gravel, crushed stone, and recycled material. They are classified in accordance with their particle size gradation, the plasticity of the fine materials, their resistance to shear stresses (stability) that causes rutting, and the durability of the particles to mechanical degradation. Caltrans uses two classes of aggregate base: Class-2 and Class-3. The quality of aggregate base material affects the extent of load distribution and drainage. The gradation of the aggregates can affect structural capacity, drainage, and frost susceptibility. A dense gradation is needed to provide structural capacity, without excessive fine materials that decrease structural capacity. The fine materials must not be susceptible to swelling and shrinking and loss of resilient modulus (stiffness) and rutting resistance when exposed to water. Aggregates, either blasted or from crushed alluvial deposits, must have sufficient angularity and rough surface texture to ensure good aggregate interlock that will provide the required structural capacity in terms of resilient modulus and rutting resistance.

662.2 Treated Base

(1) *Hot Mix Asphalt (HMA).* Dense-graded Type A Hot Mix Asphalt (HMA) is used as a base. Type A HMA is used as a base under concrete slabs for rigid pavement , and is part of the pavement surface layers for flexible surfaced pavement. .

(2) *Concrete Bases.* Concrete base (CB) and lean concrete base (LCB) are plant-mixed concrete products used as base. CB is essentially unreinforced concrete pavement, constructed with or without transverse joints, used primarily for widening rigid pavement structures that have been or will be surfaced with HMA. CB is finished in anticipation of being paved with HMA. LCB is produced with less cementitious material and allows lower quality aggregates than CB. LCB is only intended for jointed concrete pavement structures as a base beneath the PCC slabs. Concrete bases can utilize cements that develop strength and/or set faster than conventional cement. Rapid strength concrete base (RSCB) and lean concrete base rapid setting (LCBRS) have the same applications as CB and LCB but are usually specified for projects with short construction windows such as individual slab replacements. Concrete bases and lean concrete bases can be

sawcut to match the joints when used beneath JPCP. CB or LCB is not used as a base for continuously reinforced concrete pavement.

(3) *Cement Treated Bases.* Cement treated bases (CTB) are granular materials mixed with portland cement to increase their strength and stiffness. CTB can be plant-mixed or mixed on the site.

(4) *Recycled Pavement Bases.* Recycled pavement bases are materials that are constructed in place by in-place crack and seating or removing and crushing existing rigid pavement for use as an aggregate base, or pulverizing existing flexible pavement. Crushed rigid pavement is used as a granular base. Caltrans does not have a specification for in-place rubblization of rigid pavement. Pulverized existing flexible pavement can be used as a granular base or can be stabilized with a combination of foamed asphalt and cement, cement, lime, or other cementitious materials. The stabilization can be done as part of in-place recycling (IPR), or within or close to the construction site using cold central plant recycling (CCPR).

(5) *Consideration of Treated Bases in Design.*

(a) Rigid Pavement. Base type is a consideration in the design of new and rehabilitated rigid pavement, and the two alternatives considered in the JPCP design catalog are HMA Type A and LCB. HMA Type A is the only type of base used for continuously reinforced concrete pavement. CTB and treated permeable bases, particularly asphalt treated permeable base (ATPB) have been used in the past. CTB showed poor faulting and pumping performance under JPCP when the concrete transverse joints did not have dowels. ATPB was susceptible to moisture damage, and also had problems regarding early slab cracking and poor joint performance when used with shoulder edge drain systems.

(b) Flexible Pavement. Base type is a consideration in the design of new and rehabilitated flexible pavement. While combinations of aggregate base and subbase are common, treated bases are also used. Flexible pavements with cracked and seated rigid pavement below the HMA surface, chemically stabilized (cement, lime, or fly ash treated) bases below the asphalt surface layers, sometimes called semi-rigid pavement, should be designed using CalME and considering reflective cracking from shrinkage cracks in the stabilized base. Recycled pavement bases that are cement stabilized and not subjected to microcracking, appropriate curing and other practices described in the in-place recycling guidance should also be designed considering reflective cracking.

662.3 Treated Permeable Base

Treated permeable bases (TPB) provide a 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 initially provide greater drainage capacity than is needed for most applications. The standard thickness is based primarily on constructability and consideration of construction variability.

TPB is not recommended for new pavement design or reconstruction unless it is needed to provide an outlet for existing TPB layers when widening. Where excess water needs to drain through the pavement, such as when the uphill side of pavement does not allow for drainage is use sub-surface drainage to carry water to the other side of the roadway, and a TPB layer just below the HMA to drain excess water through from the uphill side of the pavement to the downhill side should be avoided wherever possible.

Erosion in CTPB (water washing away cement paste and fines) can be an issue for TPB under both flexible and rigid pavement. Research conducted in the 1990s on flexible and rigid pavement by Caltrans and by the University of California Pavement Research Center (UCPRC) indicates that the use of ATPB is highly susceptible to stripping under both pavement types. Because of these problems with TPB, the Department recommends the use of standard aggregate base (AB), along with the use of a QC/QA specification for the HMA for new flexible pavement structures, instead of ATPB, to help minimize surface permeability and TPB is not included in the rigid pavement design catalog.

Where there is an issue of maintaining continuity of an ATPB/CTPB layer in existing lanes that will drain into a new lane or widening to provide drainage through the pavement structure, the site investigation should first determine by taking cores whether or not the TPB layer is capturing water and transmitting it to the conveyance system at the edge of the pavement. TPB that has been clogged with pumped fines may no longer be permeable.

Where the TPB can be determined in the site investigation to no longer be functioning as a drainage layer in the existing lanes, continuity of the TPB in the existing lanes through the new or widened lanes is not needed. Where the TPB in the existing lanes is still capturing and conveying water, a TPB should be included in the new or widened lane. The following features are recommended when using TPB:

(1) Daylight the edges or if that is not possible use edge drains (see Figure 651.2A in Chapter 650 for edge drain details).

(2) 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.

(3) Try to use permeable backfill in shoulders around edge drains to avoid the trapping of water under the pavement (referred to as the “bathtub effect”) if the edge drain becomes clogged.

662.4 Subbase and Stabilized Subgrade

Subbases may be aggregate subbase or stabilized subgrade.

Aggregate subbase is similar to aggregate base but with less restrictive quality requirements. Because of the continual depletion of quarry aggregates, most subbases typically consist of recycled pavement and/or demolition materials, or quarry products that cannot meet the criteria for aggregate base.

Excavated soil and low quality imported borrow material can be chemically treated with a stabilizing agent to increase strength and reduce expansiveness. The most common types of stabilized soils are lime stabilized soil (LSS) and cement stabilized soil (CSS). Other soil stabilization agents include asphalt binder and fly ash or kiln dust, but these are considered experimental alternatives and are not currently supported in the Department's Standard Specifications or guidelines.

Stabilizing the soil does not eliminate or reduce the required aggregate subbase for rigid or composite pavements in the rigid pavements catalog (see Topic 623). However, for flexible pavements, stabilized subgrade is a material considered in the design.

The Guidelines for the Stabilization of Subgrade Soils in California *(https://dot.ca.gov/-/media/dot-media/programs/research-innovation-system-information/documents/f0016618-task-2201-pavement.pdf)* should be consulted to assist with the selection of the most appropriate method to stabilize soils for individual projects, and the District Materials Engineer should be contacted. The final decision as to which stabilization method to use rests with the District.

Topic 663 – Engineering Properties for Base and Subbase Materials

663.1 Selection Criteria

Different types of treated and untreated base and subbase materials have different capacities for resisting stresses and deformations imposed by traffic loads, which must be considered when determining the type and thickness of pavement foundation layers. Besides load carrying considerations, other factors should be considered, such as local availability of materials, costs, climate, and past performance of pavements on nearby projects with similar subgrades, climates and drainage conditions. The District Materials Engineer should be contacted for the latest guidance in base and subbase materials among other related engineering considerations.

Minimum aggregate base thicknesses to provide a working platform for the construction of rigid and flexible pavement are shown in Table 633.1.2. Subgrade stabilization (Topic 664) or Subgrade Enhancement Geosynthetics (Topic 665) may be used in place of the minimum thicknesses in the table.

663.2 Base and Subbase for Rigid Pavements

For rigid pavements, the capacity of base and subbase materials to resist traffic loads is considered in the design catalogs found in Topic 623. The base and subbase properties used in the development of the design catalog using the *Pavement ME* software are shown in the technical documentation for the catalog, available from the Office of Concrete Pavement.

663.3 Base and Subbase for Flexible Pavements

For flexible pavement mechanistic-empirical design, the capacity of treated and untreated base and subbase materials to resist traffic loads is considered by their resilient modulus and rutting properties, and if stabilized, by rutting and cracking-related properties. Base and subbase materials have default values for the design of new pavement using CalME shown in Table 663.3. Resilient modulus values for use in the design for rehabilitation or reconstruction of existing pavement should be determined following the Site Investigation Guide.

When stabilized soil is substituted for aggregate subbase for flexible pavements, as discussed in Index 662.4, the thickness of the stabilized soil layer is obtained from design using CalME. The resilient modulus (M_r) is determined based on unconfined compressive strength (UCS) of the stabilized material as follows:

M_r (ksi)= 0.124×UCS+9.98 for lime stabilized soil and ;

M_r (ksi)= 1.2 x UCS for cement stabilized soil

These equations are only valid for UCS of 300 psi or higher at 28 days of curing. For lime and cement stabilization, UCS is determined by different test methods, but in both cases the 28-day UCS is simulated by curing prepared samples in an oven for 7 days at 110±5°F. Refer to California Test Method 373 and ASTM D 1633 for lime and cement respectively. If test data are not available, default properties are available in CalME.

Because the stabilization of soil may be less expensive than importing and placing aggregate base material, the calculated base thickness can be reduced by increasing the stabilized soil thickness. The maximum thickness of lime and cement stabilized subgrade is 2 feet with a maximum lift thickness of 1 foot.

Topic 664 – Subgrade

Subgrade is defined as the roadbed portion on which pavement, surfacing, base, subbase, fill, or a layer of any other material is placed. It is the soil or rock material underlying the pavement structure, and unlike subbase, base and wearing course (surfacing) materials whose characteristics are relatively uniform, there is often substantial variability of engineering properties of subgrade soils over the length of a project. Since pavements are engineered to distribute stresses imposed by traffic to the subgrade, the subgrade conditions have a significant influence on the choice and thickness of pavement structure and the way it is designed. A thorough understanding of the nature and distribution of subgrade soils in any pavement project area is essential to appropriately engineer the construction, rehabilitation, or widening of a highway facility. Before beginning any project, a detailed site investigation should be performed to understand the subgrade soil conditions and engineering properties. For detailed site investigation information see the "Site Investigation Guide" and for soil characteristics refer to Topic 614 - Soils Characteristics.

664.1 Subgrade Improvement Overview

Depending on the existing soils and project design, it may be cost-effective to improve the properties of the subgrade either mechanically (in addition to compaction), chemically, or both. Subgrade improvement may also be a solution to constructability problems.

Table 663.3

Default Resilient Moduli for Bases and Subbases Used in Flexible Pavement Design

Type of Material	Abbreviation	Resilient Modulus, M_r (psi)	California R-value	Poison's Ratio, (**ν**)
Aggregate Base	AS-Class 2	30,000	50	0.35
	AS-Class 3	25,000	40	0.35
	AB-Class 2	45,000	78	0.35
	AB-Class 3	30,000	50	0.35
Asphalt Treated Permeable Base	ATPB	45,000	NA	NA
Cement Treated Base	CTB-Class A	1,508,000	NA	0.2
	CTB-Class B	1,140,000	80	0.2
Cement Treated Permeable Base	CTPB	1,100,000	NA	0.2
Lean Concrete Base	LCB	1508,000	NA	0.2
Lean Concrete Base Rapid Setting	LCBRS	1508,000	NA	0.2
Lime Stabilized Soil	LSS	$0.124 \times UCS^{(1)} + 9.98$	NA	0.2
Cement Stabilized Soil	CSS	$1.2 \times UCS^{(2)}$	NA	0.2

NOTES:

(1) UCS is the unconfined compressive strength of the lime stabilized material in psi measured according to California Test Method 373 with the modification that samples are oven-cured at 110°F ± 5°F for 7 days.

(2) UCS of the cement stabilized materials in psi measured according to ASTM D 1633, oven-cured at 100±5°F for 7 days.

Legend:
NA = No default value available
UCS = Unconfined Compressive Strength in psi (minimum 300 psi)

664.2 Mechanical Stabilization

Improving strength and stiffness, and reducing permeability, are reasons for considering mechanical stabilization. Mechanical subgrade stabilization includes the following:

(1) *Compaction*. All materials, including those that are chemically stabilized, need to be compacted to required standards. Compaction increases strength and stiffness and reduces permeability. Chemically stabilized subgrades that are not well compacted will perform poorly. Subgrade soils should be ripped to a depth of one foot, moisture conditioned and compacted to required density..

(2) *Blending*. Blending involves the mixing of materials that have different properties, such as gradation and plasticity, to form a material with characteristics that improve upon the limitations of the source materials. In most instances, blending will involve adding coarse aggregates to the finer in situ material. Less common in California is the addition of fine material to in situ sandy or coarse aggregates to fill voids and obtain a denser gradation.

(3) *Subgrade Enhancement Geosynthetics.* Subgrade enhancement geosynthetics are typically geotextiles (also called fabric), geogrids, or geocomposites such as a combination of geogrid and geotextile interlayers placed between the pavement structure and the subgrade (the subgrade is usually untreated). Geosynthetics can be used for temporary improvement of subgrade to provide a platform for equipment during construction, and/or long-term enhancement to improve the ability to sustain traffic loads distributed to the subgrade. Detailed information on subgrade enhancement geosynthetics is provided in Topic 665.

664.3 Chemical Stabilization

The most common types of stabilized subgrade are lime stabilized soil (LSS) and cement stabilized soil (CSS). Other soil stabilization agents include asphalt binder and fly ash or kiln dust, but these are considered experimental alternatives and are not currently supported in the Department's Standard Specifications or guidelines.

The Guidelines for the Stabilization of Subgrade Soils in California should be consulted to assist with the selection of the most appropriate method to stabilize soils for individual projects, and the District Materials Engineer should be contacted. The District makes the final decision as to which subgrade stabilization method to use.

For rigid pavement design, low quality in-situ subgrade soil can be improved from Type III to Type II or Type I (see Table 623.1A) by chemical stabilization to a minimum depth of 0.65 foot using an approved stabilizing agent such as lime, cement, or fly ash (fly ash is not currently supported in the Department's Standard Specifications or guidelines). Chemically treated soil samples should be tested to determine the unconfined strength of the stabilized soil. To ensure long-term stability of the subgrade during the pavement design life, the stabilized soil should achieve an initial minimum unconfined compressive strength of 300 psi. For detailed information refer to "Guidelines for the Stabilization of Subgrade Soils in California": [http://www.ucprc.ucdavis.edu/PDF/UCPRC-GL-2010-01.pdf.](http://www.ucprc.ucdavis.edu/PDF/UCPRC-GL-2010-01.pdf)

Topic 665 - Subgrade Enhancement Geosynthetic

665.1 Purpose

Subgrade Enhancement Geosynthetic (SEG) can be either a Subgrade Enhancement Geotextile (SEG_T) or Subgrade Enhancement Geogrid (SEG_G) or a combination of SEG_T and SEG_G (a geocomposite) placed between the pavement structure and the subgrade (the subgrade is usually untreated). The placement of SEG below the pavement will provide subgrade enhancement by bridging soft areas.

Subgrade Enhancement Geogrids (SEG_G) are used to provide greater confinement to granular layers placed above a weak/soft subgrade that would otherwise be provided by the stiffness of the subgrade. SEG_G mobilize its tensile strength to reinforce the subgrade soil. Subgrade Enhancement Geotextiles (SEG_T) will provide a separation function between soft subgrade (with high fines content), susceptible to pumping, and high-quality subbase or base materials. On weak subgrade, the use of selected SEG_T also provides a stabilization function (i.e., the coincident function of separation and reinforcement). As the soft soil undergoes deformation, properly placed SEG will mobilize its tensile strength properties to provide increased strength to the subgrade. Typically, woven geotextile is used where improvement in tensile strength is desired by a geotextile. Check the standard specifications for different types of available geosynthetics for SEG.

Other benefits of using SEG include:

- Potential cost savings:
  - Reduced subbase or aggregate base thickness in some situations
  - Reduced, or elimination of, the amount of soft or unsuitable subgrade materials to be removed
- Preventing contamination of the base and subbase materials with plastic fines from clay subgrades (when using SEG_T)
- Reduced disturbance of soft or sensitive subgrade during construction
- Ability to install in a wide range of weather conditions.

665.2 Properties of Geosynthetics

(1) Subgrade Enhancement Geotextile (SEG_T). Mechanical, physical, and other properties of geotextile (SEG_T) used for subgrade enhancement must meet the requirements in Section 96 of the Standard Specifications.

(2) Subgrade Enhancement Geogrid (SEG_G). Property requirements for SEG_G are related to performance. The most important geogrid properties for subgrade enhancement related to performance and durability are tensile strength, junction strength, flexural rigidity, and aperture size.

Different types of geogrid can be used for SEG_G provided their stabilizing performance is equivalent to or greater than the values specified in Section 96 of the Standard Specifications.

665.3 Required Tests

The following geotechnical soil laboratory tests are required to evaluate subgrade for geosynthetic applications:

- Atterberg Limit Tests: CT 204 or alternatively ASTM D4318 or AASHTO T 90.
- Sieve Analysis: CT 202 or alternatively ASTM C136 or AASHTO T 27.
- Resilient Modulus Test: AASHTO T 307 or alternatively Dynamic Cone Penetrometer (DCP) (ASTM D6951), R-value test (CT 301) or California Bearing Ratio (CBR) test (ASTM D1883 or AASHTO T 193). Estimated values for resilient moduli based on DCP, R-value or CBR values can be found in the Site Investigation Guide.

665.4 Mechanical Stabilization Using SEG

Subgrade enhancement geosynthetics - SEGs (SEG_T and SEG_G) achieve mechanical stabilization through different mechanisms:

(1) Subgrade Enhancement Geotextile (SEG_T). A geotextile's primary stabilization mechanism is filtration and separation of a soft subgrade and the subbase or base materials. The sheet-like structure provides a physical barrier between these materials to prevent the aggregate and subgrade from mixing. It can also reduce excess pore water pressure through a mechanism of filtration and drainage. Secondary mechanisms of a geotextile are lateral restraint and reinforcement. Lateral restraint is achieved through friction between the surface of the geotextile and the subbase or base materials. The reinforcement mechanism requires deformation of the subgrade and stretching of the geotextile to engage the tensile strength and create a "tensioned membrane." Typically, woven geotextile is recommended for reinforcement mechanism, where subgrade enhancement is desired by tensile strength.

(2) Subgrade Enhancement Geogrid (SEG_G). The primary stabilization mechanism of a geogrid is lateral restraint of the subbase or base materials through a process of interlocking between the aggregate and the apertures of the geogrid. The level of lateral restraint that is achieved is a function of the type of geogrid and the quality and gradation of the base or subbase material placed on the geogrid. To maximize the performance of the geogrid, a well-graded granular base or subbase material should be selected that is sized appropriately for the aperture size of the geogrid. When aggregate is placed over a geogrid it quickly becomes confined within the apertures and is restrained from punching into the soft subgrade and shoving laterally. This results in a "stiffened" aggregate platform over the geogrid. Very little deformation of the geogrid is needed to achieve lateral restraint and reinforcement. Separation and filtration/vertical drainage are secondary mechanisms of a geogrid. Because the aggregate is confined within the apertures of the geogrid and cannot move under load, separation and filtration can be achieved. A layer of a geotextile can be used to prevent the migration of fines into base/subbase materials or for separation purposes along with the geogrid as a geocomposite.

665.5 Selecting Geosynthetic Type and Design Parameters

(1) *Determining SEG Functions* - Subgrade stabilization is the primary function for geogrids installed between an aggregate base or subbase and the subgrade. The primary functions of geotextiles are separation, stabilization, filtration, and drainage. Subgrade soils with resilient modulus <12 ksi (equivalent R-value <20) are considered poor or weak soils and may require SEG to provide reinforcement as the primary function and separation as the secondary function. However, depending on the type and treatment of the base layer, pavements constructed over subgrade soils with resilient modulus up to 23 ksi (equivalent R-value 40) can benefit from separation if the subgrade soil contains significant percentages of fines.

(2) *Conditions for Using SEG and Selecting SEG Type* - SEG is generally selected based on the following criteria:

- On soft subgrade conditions (4.0 ksi $\leq M_r <$ 6.5 ksi), consider placing a thicker initial lift (minimum of 0.5 feet) of subbase or aggregate base material on top of the SEG to effectively bridge the soft soils and avoid bearing capacity failure under construction traffic loading.
- Use of SEG_G only is not recommended unless the aggregate material placed above the subgrade meets the following natural filter criteria: (D_{15}Aggregate Base/D_{85}Subgrade) ≤ 5 and (D_{50}Aggregate Base /D_{50}Subgrade) ≤ 25, where D_{15}, D_{85}, and D_{50} are grain sizes of the soil particles for which 15 percent, 85 percent, and 50 percent of the material is smaller than these sieve sizes. If the aggregate base material does not meet the above natural filter criteria, geotextiles that meet both separation and stabilization requirements are recommended, or a composite (combination of SEG_T and SEG_G) is recommended.
- For subgrade resilient modulus less than 15 ksi (equivalent R-value <25), SEG_G is most applicable, with or without geotextile, depending on the natural filter criteria.
- For subgrade resilient modulus between 12ksi and 15ksi (equivalent R-values between 20 and 25), SEG_G is generally selected for its stabilization function, depending on natural filter criteria. The stabilization requirements for SEG_G can be found in Section 96 of the Standard Specifications or SEG Design and Construction Guide.For subgrade resilient modulus greater than 15 ksi (equivalent R-values >25) but less than 23 ksi (equivalent R-value <40), the engineer may consider utilizing SEG_T as a separator.
- For subgrade resilient modulus greater than 15 ksi (equivalent R-values >25) but less than 23 ksi (equivalent R-value <40), the engineer may consider utilizing SEG_T as a separator.
- For subgrade resilient modulus greater than 23 ksi (equivalent R-value >40), the use of SEG may not provide any benefit.
- For very soft subgrade conditions, Mr < 4 ksi or R-value < 5 or CBR < 2.5, the subgrade should be treated as unsuitable materials. Remove and replace with

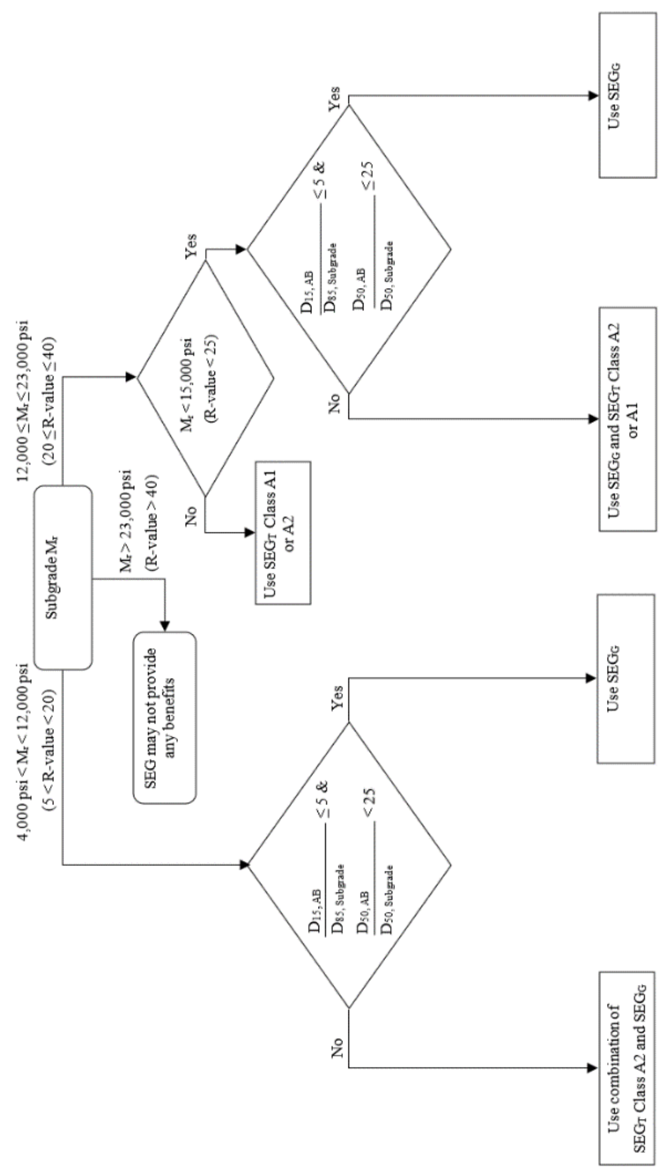
imported borrow materials or treat with lime or cement as specified in Standard Specification Section 24.

Use the flowchart shown in Figure 665.5 for the optimal selection of the most appropriate type of geotextile or geogrid based on subgrade resilient modulus, or estimated resilient modulus based on DCP, CBR, or R-value test results and gradation of the subgrade and aggregate base materials.

Before choosing to use SEG, the engineer should investigate the engineering and economic benefits of using SEG_G and/or SEG_T instead of designing the pavement for the existing subgrade resilient modulus.

Figure 665.5

Flowchart for SEG Selection



665.6 Appropriate Application of SEG

Where SEG may be the most cost-effective solution, include areas with the following soil characteristics:

- Poor (low strength) soils, which are classified in the Unified Soil Classification System (USCS) as clayey sand (SC), lean clay (CL), silty clay (ML-CL), high plastic clay or fat clay (CH), silt (ML), high plasticity or elastic silt (MH), organic soil (OL/OH), and peat (PT);
- Resilient Modulus (M_r) < 15 ksi (R-value 25), and/or other properties stated above in Index 665.5(2);
- High water table and high soil sensitivity.
- Shallow utilities or contaminated soils.

665.7 Other Design Considerations

The following should also be considered by the design engineer when designing pavements involving SEG:

- On soft subgrade soils, SEG may be used instead of stabilizing material such as lime or cement if the purpose of the treatment is solely a working platform to provide access to construction equipment. It may also be used with lime modified soil, where lime is used to modify the plasticity of the soil, but not result in permanent stabilization When placed with an aggregate subbase, SEG provides a working platform for access of construction equipment, typically on subgrade with resilient modulus 4.0ksi to 6.5ksi (equivalent R-values of 5 to 10).
- For information on how to mitigate for expansive clay subgrade with plasticity index (PI) greater than 12, see Index 614.5 Expansive Soils.
- Perform a filter analysis if the soil material types described in Index 665.5(2) are either above or below the limits shown in Figure 665.5 when SEG_G is considered to determine natural filter criteria are met, to control migration of fines into the subbase or aggregate base materials.
- For applications involving drainage and filtration, the design engineer should verify that the permeability of SEG_T is greater than the permeability of the soil.
- If SEG_T is to be placed in direct contact with recycled concrete material, SEG_T made of polyester should not be used. Otherwise, a separating layer (such as an aggregate base) with thickness greater than 0.33 feet (minimum of 4") must be placed to separate the geotextile from the recycled concrete material.
- SEG is not necessary if chemical stabilization such as lime or cement treatment of the subgrade is planned.

665.8 Subgrade Stiffness Enhancement with SEG

The use of SEG on weak subgrade (with modulus < 15 ksi or equivalent R-value < 25) can increase the effective stiffness of such soils. Therefore, the benefit of using SEG on such weak soils can be realized by 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 foundation.

The following resilient modulus values are recommended for designing SEG per Figure 665.5 on subgrade with low modulus values (less than 15 ksi [equiv. R-value 25]):

- For subgrade with a resilient modulus of equal or greater than 4 ksi (equiv. R-value 5) and less than 12 ksi (equiv. R-value 20), a design modulus value of 12 ksi can be used if SEG_G is utilized.
- When subgrade has a modulus value equal to or greater than 12 ksi (equiv. R-value 20) and less than 15 ksi (equiv. R-value 25), a design modulus value of 15 ksi can be used if SEG_G is utilized. An additional geotextile separator (SEG_T) may be used above the SEG_G to provide for the function of filtration and separation unless the aggregate base material meets the natural filter criteria presented in Index 665.5(2).

665.9 SEG Abbreviations and Definitions

The following is a list of definitions related to subgrade enhancement geosynthetics and their applications:

Apparent Opening Size: A geotextile property that indicates the approximate diameter of the largest soil particle that would effectively pass through the geotextile. Commonly, 95 percent of the geotextile openings are required to have that diameter or smaller as measured using ASTM D4751.

Aperture Shape: Describes the shape of the geogrid opening.

Aperture Size: Dimension of the geogrid opening.

D_{15}: The particle (or grain) size represented by the "15 percent passing" point when conducting a sieve analysis of a soil sample.

D_{50}: The particle (or grain) size represented by the "50 percent passing" point when conducting a sieve analysis of a soil sample.

D_{85}: The particle (or grain) size represented by the "85 percent passing" point when conducting a sieve analysis of a soil sample.

Filtration: The process of allowing water out (perpendicular to plane of geotextile) of a soil mass while retaining the soil.

Geogrid: A geosynthetic formed by a regular network of integrally connected tensile elements with apertures of sufficient size to allow "strike-through" and interlocking with surrounding soil, rock, or earth to improve the performance of the soil structure.

Geosynthetic: A group of synthetic materials made from polymers that are used in many transportation and geotechnical engineering applications.

Geotextile: A permeable sheet-like geosynthetic which, when used in association with soil, has the ability to provide the functions of separation, filtration, reinforcement, and drainage to improve the performance of the soil structure.

Grab Tensile Strength: The maximum force applied parallel to the major axis of a geotextile test specimen of specified dimensions that is needed to tear that specimen using ASTM D4632.

Nonwoven Geotextile: A planar geotextile typically manufactured by putting small fibers together in the form of a sheet or web, and then binding them by mechanical, chemical and/or solvent means.

Woven Geotextile: Produced by interlacing two or more sets of yarns, fibers, or filaments where they pass each other at right angles.

Permeability: The permeability of soil or geotextile is the flow rate of water through a soil or geotextile. The permeability of a geotextile can be determined by permittivity, which can be measured using ASTM D4491, multiplied by its effective thickness and the permeability of soil can be measured using ASTM D2434 or 5084.

Permittivity: The volumetric flow rate of water per unit cross-section area of a geotextile, per unit head, in the normal direction through a material as measured using ASTM D4491.

Puncture Strength: The measure of a geotextile's resistance to puncture determined by forcing a probe through the geotextile at a fixed rate using ASTM D6241.10.

Reinforcement: The improvement of the soil system by introducing a geosynthetic to enhance lateral restraint, bearing capacity, and/or membrane support.

Separation: A geotextile function that prevents the intermixing between two adjacent dissimilar materials, so that the integrity of the materials on both sides of the geotextile remains intact.

Stabilization: The long-term modification of the soil by the coincident functions of separation, filtration, and reinforcement furnished by a geosynthetic.

Tear Strength: The maximum force required to start or to propagate a tear in a geotextile specimen of specified dimensions using ASTM D4533.

Ultraviolet Stability: The ability of a geosynthetic to resist deterioration from exposure to the sun’s ultraviolet rays as tested using ASTM D4355.

Topic 666 – Foundation Mechanistic-Empirical Parameters for Flexible Pavements

666.1 Layer Stiffness

(1) Use. The mechanistic-empirical (ME) method for flexible pavement design requires the stiffness and its variability for each layer. In CalME, different stiffness models are used for different layers depending on the material type:

- For cement bound materials, layer stiffness is affected by damage caused by traffic. This is the case for CTPB, CTB, CB, LCB, LSS, CSS, FDR-C, FDR-L, CCPR-C, and CCPR-L.
- For asphalt bound materials, layer stiffness is a function of layer temperature and traffic speed, and damage caused by traffic. The function is typically referred to as the stiffness master curve. This is the case for HMA, FDR-FA, PDR-FA, PDR-EA, CCPR-FA, and CCPR-EA. Although they use the same basic models in CalME, FDR and PDR have different coefficients than HMA reflecting that they are not as completely and continuously bound by the asphalt binder as HMA.
- For unbound materials, layer stiffness is a function of over-burden confinement and the magnitude of axle load. This is the case for aggregate base, aggregate subbase, FDR-N (no stabilization) and all subgrade.

For each stiffness model, there is a parameter called the reference stiffness that represents the typical loading condition and controls the overall magnitude of the stiffness.

In CalME, each material in the built-in Standard Materials Library has an applicable stiffness model. Each material has default values for the reference stiffness and its variability.

Table 614.3 provides default reference stiffnesses for subgrade soils based on their classification using the Unified Soils Classification System along with their Poisson's ratios (v). Refer to Table 614.2 for the Unified Soil Classification System.

Table 663.3 provides default reference stiffnesses of the bases and subbases typically encountered in constructing flexible pavements along with their Poisson's ratios (v), a parameter also required for design using the ME methods.

The default values should be overwritten by values determined through site investigation when applicable (refer to the Site Investigation Guide).

(2) Determination. The layer stiffness in a pavement can be determined by several means depending on project size, and whether the design is for new construction, reconstruction, or rehabilitation.

- In the laboratory, the resilient modulus of an unbound material is measured under a variety of conditions simulating the physical (e.g., moisture, density, etc.) and stress state conditions of the material subjected to moving wheel loads. Experimentally, it is determined from a relationship between stress and deformation of the material derived using a modified repeated load triaxial testing machine. The loading device in this specialized automated machine is capable of applying repeated cycles of haversine-shaped load pulses of 0.1 second duration followed by a rest period (0.9 seconds for hydraulic loading devices and 0.9-3.0 seconds for pneumatic loading devices) in accordance with the procedure described in AASHTO T 307, (Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials). Numerically, it is calculated as the ratio of applied deviator stress (vertical stress less confining pressure) to recoverable or resilient strain. The resilient modulus determined using this procedure represents the elastic modulus of the tested materials which recognizes certain nonlinear characteristics. The resilient modulus derived from experiments conducted on material samples could be used in designing a flexible pavement using the ME design method.

- In the field, the following methods (when appropriate) can be used either independently or in combination to determine layer stiffness:
  - FWD deflection testing followed by the back-calculation analysis described in Index 635.4 can be used to determine layer stiffnesses for all materials. For detailed information refer to CT 357 and the "Site Investigation Guide" *Need to provide a link*
  - Dynamic cone penetrometer (DCP) tests can be used to estimate layer stiffness through the correlation between penetration rate and soil stiffness.
  - USCS classification of unbound materials can be used to estimate typical layer stiffness.

Refer to the Site Investigation Guide for more details.

666.2 Rutting Resistance

(1) Use. The mechanistic-empirical (ME) method for flexible pavement design requires rutting resistance model parameters. In CalME, different rutting models are used for different layers depending on the material type:

- For cement and lime stabilized materials, rutting is negligible. This is the case for CTPB, CTB, CB, LCB, FDR-C, FDR-L, CCPR-C, CCPR-L, LSS, and CSS.
- For asphalt bound materials, rutting is dependent on the shear stress caused by the traffic load. Rutting is only calculated in HMA layers that are within 0.33 ft of the pavement surface, where temperatures and shear stresses are high enough to cause rutting. This is the case for HMA, FDR-FA, PDR-FA, PDR-EA, CCPR-FA, and CCPR-EA.
- For unbound materials, rutting is dependent on the vertical strain at the top of the layer. This is the case for aggregate base, aggregate subbase, FDR-N, and all subgrades.

In CalME each material in the built-in Standard Materials Library has an applicable rutting model. Each material has corresponding rutting model parameters. It is important to select the appropriate material type for each layer. For subgrades, the material type is defined by the USCS classification.

CHAPTER 680 – PAVEMENT DESIGN FOR WIDENING PROJECTS

Topic 681 – Pavement Widening Overview

Index 681.1 – Background

(1) *Purpose* - Pavement widening involves the construction of additional width to improve traffic flow and increase capacity on an existing highway facility or to improve existing features such as the inclusion of shoulders, turn lanes, and passing lanes. Pavement widening projects create unique issues for pavement engineers such as what is the best structure to build for the widening and how to make the widened pavement or new lanes compatible with existing pavement. This Chapter provides basic instructions and guidance for selecting pavement type, design standards, and details for pavement widening projects.

(2) *Types of Pavement Widening Projects* - Pavement widening may involve the following types of pavement projects:

- Adding travel lanes (including bus or bicycle lanes), auxiliary lanes, climbing or passing lanes
- Adding shoulders, pullouts for maintenance/transit traffic
- Widening existing lanes, shoulders or pullouts

When planning widening projects such as lane or shoulder additions, the existing adjacent pavement condition should be investigated to determine if the measures discussed in Index 682.3 are needed to combine rehabilitation or pavement preservation work with widening.

Topic 682 – Design Considerations

682.1 Standards

Besides pavement engineering discussed in Chapter 610, pavement widening presents additional challenges in pavement design. These include achieving the following standards:

- Provide a uniform foundation across the new and existing pavement structure to accommodate both pavement drainage and fatigue performance.
- Ensure that existing pavement is adequate to sustain traffic loads expected during the design life of the new pavement widening structure. Existing pavements may have been designed decades earlier for less traffic, and those thicknesses may not only be less than those of the newly widened pavements but may also have worn surfaces and in some cases exhibit minor or major structural distress.

- Maintain functionality of the existing pavement structure drainage system.

These issues can affect the service life and drainage of both the existing and new pavement structures. To ensure both drainage and design life standards are met, the drainage conditions and structural capacity of both the existing and new pavement structures should be evaluated and the results of the evaluation should be taken into account during the planning and scoping phases of a widening project.

682.2 Pre-Design Evaluation

The following pre-design evaluations are recommended to ensure that pavement widening projects are designed and constructed to meet the performance standards described in Item 682.1.

- Perform an official investigation following the Site Investigation Guide, including:
  - Review as-built records of the existing pavement structure such as material types and layer thicknesses, and where available the as-built material properties and mix designs. Look for material types and layer thicknesses from historical cores in iCore and the Ground Penetrating Radar (GPR) data.
  - Review the current pavement condition survey data and history of past maintenance and rehabilitation treatments of adjacent lanes
- Conduct a site investigation following the Site Investigation Guide. The investigation will produce information needed for mechanistic-empirical design for flexible pavement alternatives and for selection of rigid pavement design alternatives from the catalog in Chapter 620, including:
  - Existing subgrade type and condition and pavement layer types, thicknesses, and conditions.
  - Drainage and moisture conditions in the existing pavement and areas to be widened, and condition of any drainage layers in the existing pavement that must be considered in the widening. Note that widening may cover existing drainage systems, especially side drains, and new systems will need to be designed and constructed to accommodate the additional water catchment of the wider road surface.
  - Condition, pavement structure, and subgrade stiffness in the adjacent lanes, if this information will be used in the design of the new lanes, or if the adjacent lanes will be rehabilitated as part of the widening.

682.3 Pre-Design Considerations

The following pre-design considerations are recommended when designing a pavement widening project.

(1) *Consistent and Cost-effective Overall Pavement Structure.* The engineer needs to consider what characteristics are important for both the new and existing pavement to provide a consistent, cost-effective, and functioning structure for the overall pavement. This includes considering how the new pavement will perform as well as doing a life-cycle cost analysis that considers initial cost, maintenance and rehabilitation schedules, and costs for different alternatives. The life-cycle cost analysis is to follow the Life Cycle Cost Analysis Procedures Manual (https://dot.ca.gov/programs/maintenance/pavement/concrete-pavement-and-pavement-foundations/life-cycle-cost-analysis).

(2) *Rehabilitation or Pavement Preservation of Existing Pavement with the Widening Project.* It is often not cost-effective nor desirable to widen a highway without correcting ride quality and structural distress in the adjacent pavement structures when that work is needed. During planning and scoping of widening projects, it is necessary to thoroughly evaluate the existing adjacent pavement structure to determine if rehabilitation or pavement preservation is needed in conjunction with the widening. This involves a review of the current pavement condition survey data, future projections of distress and roughness condition, and the information from the site investigation of the existing roadway. The review should be done during the project initiation phase and updated during the design phase because the pavement condition may have deteriorated during the intervening time. If rehabilitation or pavement preservation is warranted, combining rehabilitation or pavement preservation work with widening is strongly encouraged.

(3) *Future Traffic Delay and Long-Term Costs.* Combining widening with rehabilitation or pavement preservation work on existing pavement can minimize future traffic delay and long-term costs, and reduce overhead costs of managing two separate projects. If the adjacent existing lane warrants rehabilitation, the lane should be rehabilitated in conjunction with the widening and brought up to the same life expectancy as the new widened portion of the roadway (see Index 612.3. In certain circumstances, the District may defer the pavement rehabilitation work and program it as a separate project, but this should be done in coordination with Headquarters Pavement Reviewers and the Project Delivery Coordinators (for non-delegated projects per the District Design delegation Agreement). If the adjacent lane does not need to be rehabilitated, an appropriate pavement preservation treatment should be applied to provide a uniform surface for existing and widened sections and synchronization of future preservation of the existing and new pavement.

Pavement preservation and rehabilitation work that should be included with widening projects for concrete and asphalt pavements are discussed in Index 682.5(2) and 682.5(3) respectively.

682.4 Design Considerations

Design of the widened lanes should follow the same procedure as new pavements. See Index 633 for design of new flexible pavements. See Chapter 620 for design of new rigid pavements. The inputs needed for new flexible and rigid pavements should be developed following the Site Investigation Guide.

682.5 Scoping, Estimating, and Detailing

The following design criteria are provided to aid in scoping, estimating, and detailing pavement widening projects. As per Index 682.1, these requirements should be viewed as minimum criteria for determining how much work to do on existing pavements. Because each widening project has different pavement engineering and performance issues, early and frequent involvement of Headquarters Pavement Designer or Reviewers is recommended to appropriately address what features to include and how to ensure the following design criteria are met.

(1) *Pavement Structure Requirements.* The following minimum requirements should apply when designing pavement structures for widening projects:

(a) If a widening project will result in any traffic lanes to be placed partially on existing pavement and partially on new pavement, then the engineer should ensure that the pavement type and structure are consistent across the lane. Avoid creating lanes that are partially asphalt, concrete, or composite because they will develop distresses at different rates and will have different distresses which will increase future maintenance costs and worker exposure to traffic. Longitudinal joints between different pavement types in the lane can also lead to more rapid deterioration of the pavement. The partial existing lane can be removed and replaced with a complete lane of new pavement, or new pavement of the same type can be added to the partial lane.

(b) Criteria for leaving the existing structure to serve as part of the new lane are:

(1) The structural capacity of the existing pavement is assessed using the future design truck traffic plus the truck traffic that has been applied to it since initial construction or the last rehabilitation using CalME or the rigid pavement design catalog (or Pavement ME), and the structure is found to be able to carry more than 90 percent of the predicted need of the combined past and future traffic,

(2) The existing pavement is in good condition as identified in the pavement condition survey, and

(3) The widening project is adding less than 2 lanes and the width of existing pavement to remain in the proposed lane is 3 feet or more.

If the existing lane meets all these criteria then the proposed widened pavement structure should match the existing pavement and, where needed, a preservation treatment applied as discussed in Index 682.5(2)(c) and (3)(b). If the existing pavement does not meet all these criteria, then it is preferable to construct new lane(s), or partially reconstruct the existing lane, to new construction standards and remove existing pavement or pavement layers to accommodate a structure that will carry the predicted future traffic. If the existing pavement is inadequate, remove the existing pavement to the lane line of the existing adjacent truck permitted lane and replace it with new pavement or partially reconstructed pavement.

(2) *Details for Widening Next to Existing Concrete Lanes.* The following design standards should apply when widening concrete roadways:

(a) Place longitudinal joints at the locations of proposed lane lines (or ultimate lane lines if project is an interim stage of an ultimate project) except as noted below:

(1) For new outside non-truck permitted lanes next to existing outer lanes and for new median lanes next to existing median lanes, place the longitudinal construction joint between the existing pavement and the new widened section at the lane line as shown in Figure 682.4A.

(2) Additional requirements and details for tying adjacent concrete slabs can be found in Index 622.4 and the Standard Plans.

(3) When existing longitudinal joints and proposed or ultimate lane lines do not align, it is preferable to construct longitudinal pavement joints between new and existing concrete (particularly isolation joints) in non-truck permitted lanes rather than truck permitted lanes.

(b) Do not place or leave slabs less than 8 feet wide in truck permitted lanes or joints within 2 feet of wheel paths. The reduced width of the slab will result in joints in the wheelpaths which will lead to early cracking of the pavement.

(c) When widening contiguous to the concrete pavement in good condition, a pavement preservation strategy in conjunction with widening is recommended if warranted, including grinding the existing rigid pavement where warranted by roughness. This provides a smooth riding surface and can eliminate old striping and pavement markings. Grinding the lane next to the proposed widening is required when the existing International Roughness Index (IRI) exceeds 90 inches per mile in order to provide a smooth platform for the paving machine to construct the adjacent pavement structure. Pavement preservation strategies are discussed in Index 603.3 and in the *Concrete Pavement Guide* (*https://dot.ca.gov/programs/maintenance/pavement/concrete-pavement-and-pavement-foundations/concrete-pavement-guide*). Additional information on procedures for concrete pavement preservation can also be found in Topic 624.

(d) Where existing concrete pavement will require rehabilitation within ten years, the widening project should consider future compatibility of the proposed structure in the widening project with the eventual concrete pavement rehabilitation strategy in the existing lanes. Pavement rehabilitation strategies are discussed in Index 603.4 and procedures for concrete pavement rehabilitation can be found in Index 625.1.

(e) Drainage continuity may require constructing the top of the subgrade for the widening at the same or lower elevation than the existing subgrade and extending underdrains from the edge of the existing pavement to an outlet beyond the new pavement structure.

(3) Details for Widening Next to Existing Flexible Lanes. The following design standards should apply when widening next to existing flexible pavement lanes. The following alternatives should be considered and evaluated in terms of life cycle cost, and other considerations:

(a) *Flexible Pavement Alternative*. Design the new pavement structure for the widening as a new flexible pavement. The final surface elevation of the new structure should match the elevation of the existing pavement. If the existing adjacent lanes have a preservation treatment or rehabilitation in the widening project, then the new widening surface elevation should match the new surface in those lanes. When widening flexible pavement, continuity with the existing pavement should be provided whenever it is economically feasible. At a minimum, the design should use compatible materials and provide for adequate drainage underneath the existing pavement.

(b) When widening adjacent to existing asphalt pavement that is in good condition, a pavement preservation strategy in conjunction with widening such as placing a non-structural wearing course over the widening and existing pavement should be done. This provides a surface with a uniform appearance, a surface course with equivalent future maintenance requirements, a clean surface for new striping configurations, as well as elimination of pavement joints which are susceptible to water intrusion and early fatigue failure.

If the asphalt concrete surface course required for the new pavement structure is thicker than the surface course in the existing lanes, the existing shall be overlaid a minimum of 0.15 feet to match the top surface of the new asphalt concrete layer.

If the existing pavement exhibits oxidation, raveling, or minor cracking, it is recommended to mill 0.15 foot of the existing asphalt surface and overlay across the entire existing pavement and the new section as shown in Figure 682.4C.

Pavement preservation strategies are discussed in Topic 634 and in the Maintenance Technical Advisory Guide (MTAG) (https://dot.ca.gov/programs/maintenance/pavement/mtag). Additional information on procedures for asphalt pavement preservation can also be found in Index 603.3.

For existing asphalt pavement that needs rehabilitation work because of major distress, the widening project should include an appropriate pavement rehabilitation strategy for the existing pavement structure at least in the lane adjacent to the widening. In such cases, project scoping and other engineering decisions consider life cycle cost as well as other project considerations such as traffic safety to determine whether pavement rehabilitation of the existing roadway should be included with the pavement widening project. If the existing flexible pavement adjacent to the widening has extensive cracking, in-place recycling should be considered as one of the alternatives for the widening project and compared with other alternatives and considerations. See the In-Place Recycling Guidance for more details. Care must be taken that the site investigation considers both the existing pavement and the subgrade in the area to be widened. See the Site Investigation Guidance for more details. Pavement rehabilitation strategies and procedures for flexible pavement rehabilitation can be found in Index 603.4.

(c) When widening asphalt pavement, continuity with the existing pavement should be provided whenever it is economically feasible. At a minimum, the design should use compatible materials and provide for adequate drainage underneath the existing pavement. This may require constructing the top of the subgrade for the widening at the same or lower elevation than the existing subgrade and extending underdrains from the edge of the existing pavement to an outlet beyond the new pavement structure.

To provide a new uniform surface for the widening and existing pavement, mill and replace 0.15 foot of the existing asphalt surface course. If the new asphalt concrete surface course required for the new pavement structure is thicker than the existing surface course in the existing lane, then the existing lane shall be overlaid with a minimum of 0.15 feet of the same material to be used in the widening so that the surfaces of the new and existing lanes match. Figure 682.4B shows a typical pavement widening structure adjacent to existing previously cracked, seated and asphalt overlaid concrete pavement.

(d) Widening of asphalt roadways with concrete lanes should not be done except if both are true:

(1) Concrete pavement will be placed across all the truck permitted lanes, or there is a funded project to replace or overlay the existing lanes with concrete within the next 10 years.

(2) The concrete pavement joint will be located at the proposed lane line (or ultimate lane line if the project is just an interim stage of an ultimate project.)

(4) Drainage of Pavement Widening Structure. Perpetuate pavement drainage in accordance with Chapter 650. The pavement structure of the widening should be designed where feasible to provide a path for subsurface water drainage to the edge of pavement. If it is not feasible to accomplish this, then consult with Headquarters Pavement Reviewers for other options.

682.6 Other Considerations

In addition to the foregoing design considerations, the following measures should be considered when constructing a pavement widening project.

(1) Geosynthetic Interlayer at Joint between Existing Pavement and Widening. Consideration should be given to using a geosynthetic pavement interlayer at the longitudinal joint between new and existing pavement prior to applying the full-width overlay to delay reflective cracking of the joint.

(2) *Selection of the Base Material.* Selection of the new base material should be based on laboratory evaluation of both new and existing materials to compare the moisture susceptibility and drainage characteristics of each. Preferably, the moisture susceptibility and permeability of the existing and new base materials should be similar. A lower permeability or more moisture susceptible material should not be used for base, subbase, or fill material where it would block the ability of water to drain to the edge of the pavement or draw water to itself. Consideration should be given to the potential for water ingress at longitudinal joints at the interface between widening pavements that are of a different type than the adjacent existing pavement.

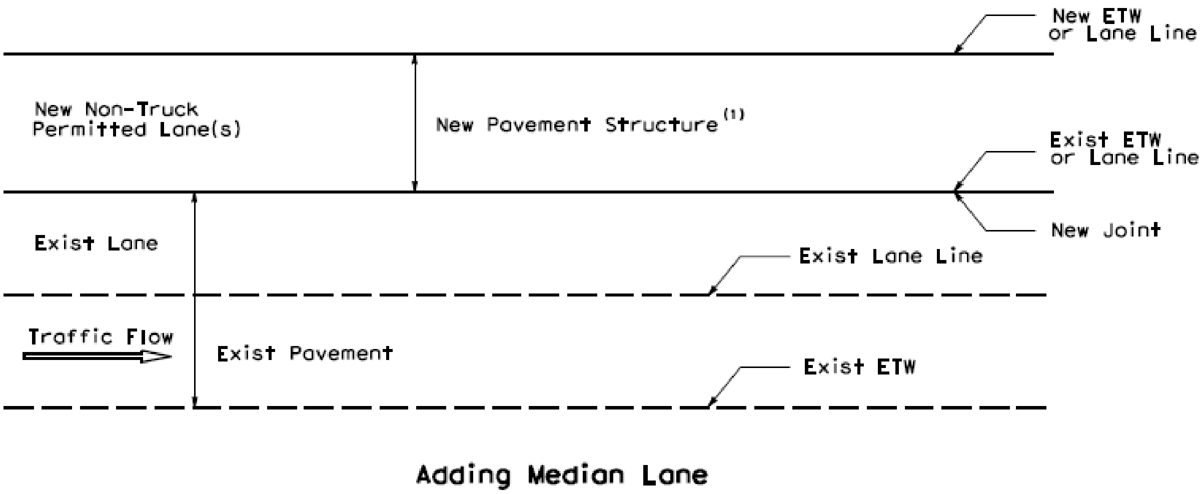
(3) *Treated Base Sections.* Other considerations will closely parallel those discussed in Index 662.2 for treated base materials. There are cases where it may be desirable to use full-depth HMA for the widening to expedite construction, even though the base for the existing pavement was cement-treated material. This strategy should not cause subsurface moisture flow problems ("bath tub" effect) provided that the cement treated base is not moisture susceptible. Laboratory evaluation of core samples will determine the degree of moisture susceptibility of the existing base.

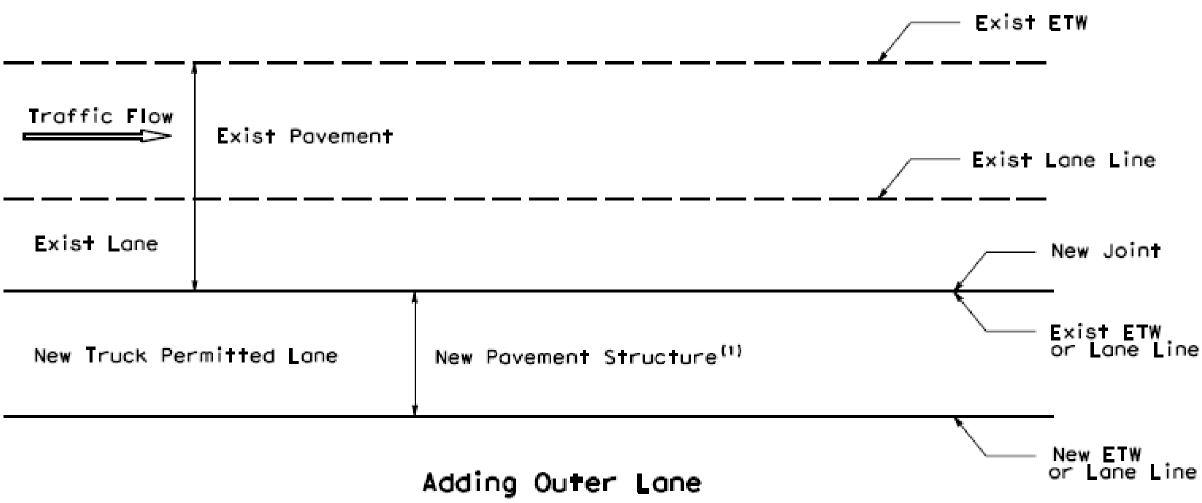
682.7 Life-Cycle Cost Analysis for Widening Projects

In addition to selecting the type of pavement for the widening project, as discussed in Topic 619, life-cycle cost analysis is a key component in determining how best to maintain both new and existing pavements over time and whether it is better to design the widening to match the life of the existing pavement or plan for the upgrading of the existing pavement to match the design life of the new pavement. When doing a life-cycle cost analysis for pavement widening, it is often best to evaluate the best alternative for upgrading the structural capacity of the existing pavement to meet current design life standards first, since the type and condition of the existing pavement will often influence the engineering of the new pavement. Life-cycle cost analysis is discussed further in Topic 619 and the Life-Cycle Cost Analysis Procedures Manual (*https://dot.ca.gov/programs/maintenance/pavement/concrete-pavement-and-pavement-foundations/life-cycle-cost-analysis*).

Figure 682.4A

Typical Concrete Pavement Widening Median Lane and Outer Lane



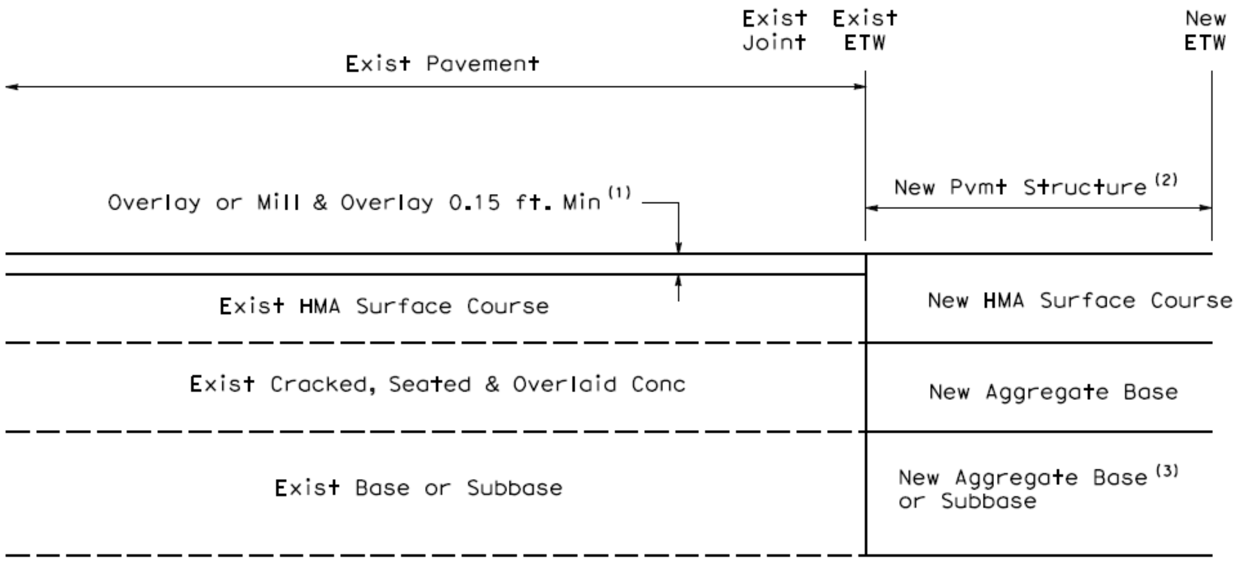


NOTES:

(1) See Index 623.1 and Tables 623.1B – M for details on concrete pavement structure design.

Figure 682.4B

Widening Previously Cracked, Seated, and Overlay Concrete Pavement in Good Condition



NOTES:

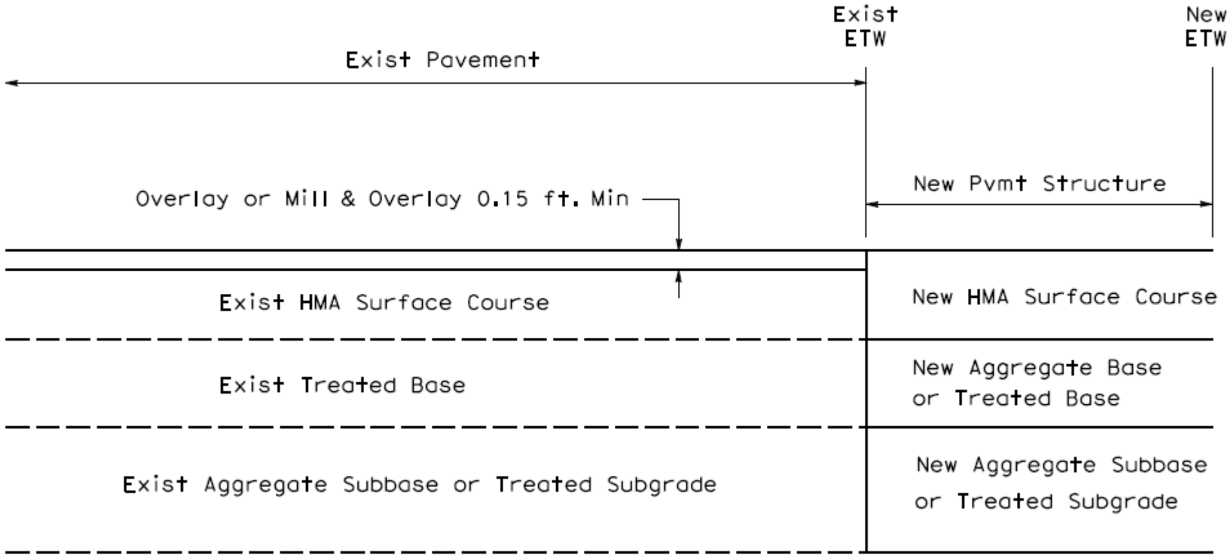
(1) See Figures 682.4A for additional details.

(2) Match the structural layers of the existing pavement for situations described in section 682.4 (1)(a).

(3) When matching existing treated base, granular base/sub base, or adding structural capacity.

Figure 682.4C

Widening Asphalt Pavement in Good Condition



objectionable. If it is concluded that the objection is valid, a more compatible facility may be substituted, subject to the following controls:

- Preference should be given to retaining the standard fence along the ramp to the end of the curb return or beginning of the taper on the local road. Where this is not reasonable, there may be substituted a fence or wall of equal or better durability and utility that is at least 4 feet high relative to the grade of freeway right of way line. Walls, ornamental iron fences with closely spaced members, or chain link fences are examples of acceptable possibilities.
- Along the local road, beyond the end of the curb return or the beginning of the taper, a facility of somewhat lower standards may be employed, if considered appropriate. The minimum allowable height is 2.5 feet above the grade at the edge of the right of way. In addition to the fence types suitable for use along the ramp, split rail fences, wooden picket fences, and permanent planter boxes are examples of possibilities. The intent is to delineate the access control line and discourage access violations in an effective manner.
- Generally, all costs for the removal of the existing freeway fence and the installation and future maintenance of a nonstandard fence are to be the property owner's responsibility under the terms of the encroachment permit authorizing the substitution. On new construction, the property owner is to assume similar costs and responsibilities subject to a credit for the value of a standard fence.

(4) Location of Fences. Normally, fences on freeways should be placed adjacent to, but on the freeway side of the right of way line.

Fences in the outer separation normally should be placed so that the area outside of the fence may be relinquished to the local agency. Typical fence placement should be at the freeway right of way line per Figure 307.4B.

When viewed at a flat angle, chain link fencing restricts sight distance. This fact should be considered in the location of such fencing at intersections. To eliminate hand maintenance, right-angle jogs should be avoided.

(5) Locked Gates. Locked gates may be provided in access control fences in special situations. A proposal for a locked gate must address a necessity. Although openings controlled by locked gates do not constitute access breaks in the usual sense of access control, they must be shown on the plans. When locked gates are proposed there must be a specific reason for each gate. All gates must be kept locked and secured. Locked gates fall into two categories:

(a) Locked gates to be used exclusively for access by highway maintenance forces on Interstate require FHWA approval. Locked gates to be used exclusively for access by highway maintenance forces on non-Interstate may be approved by the District Director. The integrity and security of this access must always be assured. Maintenance forces must also keep gates locked when not being used for the access of persons or equipment. When locked gates are to be used exclusively by highway maintenance forces, one or more of the following criteria apply:

- A circuitous route would be eliminated.
- The gate access would minimize the exposure of maintenance workers to highway traffic.
- Parking is available outside the gate.
- The gate would allow slow moving equipment to be kept off the highway.

- The site is not accessible to maintenance personal or equipment from the freeway.

(b) Proposals for locked gates to be used by utility companies, non-utility entities, or public agencies on non-Interstate must be submitted to the District Director for approval. The gate submittal must present all pertinent facts and alternate solutions.

Locked gates to be used by utility companies, non-utility entities, or public agencies require FHWA approval if the gate is on an Interstate route.

When proposals for locked gates requiring FHWA approval are included in the plans for new construction, including landscaping projects, FHWA approval of such gates will be included in FHWA approval of the project PS&E. Subsequent installations requiring FHWA approval must be submitted separately to FHWA by the Division of Design after recommendation for approval by the Chief, Office of Project Support, Division of Design.

701.3 Private Fences

(1) Placement. Caltrans will construct or pay the cost of fences on private property only as a right of way consideration to mitigate damages. Caltrans' construction of such fences should be limited to:

(a) The reconstruction or replacement of existing fences.

(b) The construction of fences across property that had been previously enclosed by fences.

These criteria apply to all private as well as public lands.

(2) Private Fences Inside the State Right of Way. Private fences may be constructed within the State right of way via Encroachment Permit to restrict access to facilities (e.g., canals) crossing under or through Department-owned property. A Maintenance Agreement must be executed to provide for future maintenance of the fence and allow access to the private utility.

701.4 Temporary Fences

(1) Placement. Temporary fences are located where necessary in accordance with construction contractor activities and where the right of way rights have been acquired.

(2) Types of Fences. Temporary fence design should conform to the needs of the situation and the length of time to be used. In most access control or demarcation applications the fence fabric will conform to permanent fence standards, while lesser requirements may apply to posts and post footings to more readily accommodate removal when no longer needed.

Temporary fence used during reconstruction of private fences must be of a type adequate to meet the permanent private fence purposes.

701.5 Other Fences

(1) ESA and Species Protection Fences. District Environmental Unit staff must specify the required placement limits and locations for ESA and species protection fences.

ESA fence material requirements are described in Section 14 of the Standard Specifications.

Species protection fences will be uniquely designed to meet the needs of the target species. District Environmental staff will provide information on the necessary design parameters. In many instances, species protection fence will be able to be directly attached to existing freeway or expressway access control fence and thus preclude the need for separate posts. Where species protection fence is to be constructed along conventional highways, it must be constructed inside the State right of way and should not be attached to any private fence that may exist.

(2) *Enclosure Fences*. Because these fences are commonly intended to provide security for Caltrans facilities, the facility type and location will often dictate the fence design to be used. Standard chain link (CL-6) fence is most common, but additions (barbed wire extension arms) or alternative designs may be considered. When slats are included as an element of the design, wind forces are considered and a resulting increase in the size and depth of embedment of fence posts as well as an increase in the size of the concrete footing occurs. See the Standard Plans for further details including post size and footing dimensions for various fence heights.

Typically District Maintenance or Traffic Operations will specify any unique design requirements for enclosure fences as they will assume responsibility after construction.

Topic 702 – Miscellaneous Traffic Items

702.1 References

(1) *Guardrail and Crash Cushions.* See Traffic Safety Systems Guidance.

(2) *Markers.* See Part 3 of the California Manual on Uniform Traffic Control Devices (California MUTCD).

(3) *Truck Escape Ramps.* See Traffic Bulletin No. 24, (1986) and the NCHRP Report 178.

(4) *Mailboxes.* See the AASHTO Roadside Design Guide, 3rd Edition, Chapter 11, "Erecting Mailboxes on Streets and Highways."

Topic 703 – Special Structures and Installation

703.1 Truck Weighing Facilities

The Division of Traffic Operations coordinates the design and construction of truck weighing facilities with the California Highway Patrol in Sacramento. Typical plans showing geometric details of these facilities are available from the Headquarters Division of Traffic Operations. Districts should refer truck weighing facility maintenance issues to their District maintenance units.

See Index 107.1 for additional details on roadway connections for truck weighing facilities.

703.2 Rockfall Restraining Nets

Rockfall Restraining Nets are protective devices designed to control large rockfall events and prevent rock from reaching the traveled way. The systems consist of rectangular panels of woven wire rope vertically supported by steel posts and designed with frictional brake elements capable of absorbing and dissipating high energies. For additional information on the characteristics and applications for rockfall restraining nets, designers should contact the Division of Engineering Services - Geotechnical Services (DES-GS).

Topic 704 – Contrast Treatment

704.1 Policy

In general, delineation should be composed of the standard patterns discussed in Part 3 of the California MUTCD.

Markings include lines and markings applied to the pavement, raised pavement markers, delineators, object markers, and special pavement treatments.

Contrast treatment is designed primarily to provide a black color contrast with an adjacent white surface. Normally, contrast treatment should be used only in special cases such as the following:

(a) To provide continuity of surface texture for the guidance of drivers through construction areas.

(b) To provide added emphasis on an existing facility where driver behavior has demonstrated that standard signs and markings have proven inadequate.

When contrast treatment is applied, a slurry seal should be used.

See Part 3 of the California MUTCD for additional information on contrast treatment.

Topic 705 – Materials and Color Selection

705.1 Special Treatments and Materials

Special materials or treatments, such as painted concrete, or vinyl-clad fences, are sometimes proposed for aesthetic reasons, or to comply with special requirements.

The following guidelines are to be used for the selection of these items:

(a) Concrete should not be painted unless exceptional circumstances exist, due to the continuing and expensive maintenance required. Concrete subject to unintentional staining should be textured during construction to minimize the visibility of stains, if other methods of controlling stain-producing runoff or dripping cannot be accomplished.

(b) Vinyl-clad fences are sometimes specified for aesthetic reasons. The cost of this material is higher than that of galvanized steel. Special consideration should be given to the life-cycle cost and maintainability of vinyl-clad fencing prior to selection for use. The use of black or green vinyl-clad mesh for access control fencing, safety fencing at the top of retaining walls, and pedestrian overcrossing fencing is acceptable.

705.2 Colors for Steel Structures

Colors for steel bridges and steel sign structures may be green, gray, or neutral tones of brown, tan, or light blue.

Criteria for selection of colors are:

(a) General continuity along any given route.

(b) Coordination of color schemes with adjacent Districts for interdistrict routes.

(c) Requests from local agencies for improvement of aesthetics in their community.

Color selection for steel bridges should be mutually satisfactory to the Division of Engineering Services and the District. The Division of Engineering Services (DES) will initiate the color selection process by submitting the proposed color to the District Landscape Architect for review. The color for steel sign structures will be selected by the District Landscape Architect.

Topic 706 – Roadside Management and Vegetation Control

706.1 Roadside Management

Consider the full life-cycle cost of transportation improvements including the long-term cost of maintenance. The design alternative with the lowest initial construction cost may not be the best solution if this approach will include high recurring maintenance costs. Designers should strive to select design approaches that do not require extensive recurring long-term activities.

The design should contribute to the safety of Department maintenance workers by incorporating techniques that eliminate or reduce worker exposure to traffic. See Index 901.2.

The following conditions must be considered in projects:

- Guardrail, including standard railing, terminal system end treatments, guard railing at structure approach and departures, and at fixed objects should include vegetation control. For more detailed information regarding placement of vegetation control consult with both the District Landscape Architect and District Maintenance. See the Standard Plans for vegetation control.
- Thrie beam barrier, including single thrie beam barrier, double thrie beam barrier, at structure approach and at fixed objects should include vegetation control. For more detailed information regarding placement of vegetation control consult with both the District Landscape Architect and District Maintenance. See the Standard Plans for vegetation control.
- Unpaved narrow strips often result from the construction of noise barriers or concrete barriers beyond the paved shoulder edge. Unpaved strips 15 feet or less in width, parallel and immediately adjacent to the roadway, should be paved to the barrier or wall. Paving these areas eliminates the need for manual vegetation control, and allows automated equipment to remove litter and debris. Pavement requirements are consistent with the guidance contained in this manual. Contrasting surface treatment such as markings, delineation, or color may also be provided so drivers can distinguish these areas from those intended for vehicular use. Consult with the District Landscape Architect for contrasting surface selection.
- Unpaved areas greater than 15 feet in width may include vegetation control techniques such as weed control mats, patterned asphalt or stamped concrete paving, or the planting of low maintenance vegetation such as native grasses. Consult the District

Landscape Architect and District Maintenance to select an appropriate vegetation control technique.

- Noise barriers should be designed with a textured aesthetic treatment and/or planted with vines to reduce maintenance required to control graffiti. Index 904.7 contains information of the planting on noise barriers.
- Unpaved area beyond the gore pavement should be paved as per Index 504.2(2).
- When placing roadside facilities that require recurring maintenance, the designer should strive to include improvements that facilitate safe maintenance access such as maintenance vehicle pullouts, maintenance access paths, walk gates and vehicle gates. It is preferred that access be provided from outside the right-of-way for all facilities that require maintenance access.
- When placing noise barriers in areas with a narrow right of way, the designer should consider locating a concrete safety shape barrier 3 feet from the face of the noise barrier to provide protected maintenance access.

Formal safety reviews for roadside management issues should be accomplished as discussed in Index 110.8. Consult the District Landscape Architect and District Maintenance unit early during design development to identify and address potential roadside management issues, such as avoiding the redundant placement of roadside facilities, or allow for the consolidation of roadside facilities.

706.2 Vegetation Control

Weed control fabric or preemergent chemicals may be placed under pavement to prevent weed growth through medians, traffic islands, and other paved areas.

The Division of Maintenance is responsible for the selection of herbicides. Approval is required for any changes from the currently approved Standard Specifications and Standard Special Provisions for pesticides and herbicides.

Since preemergents may be transported by water, they should be mixed with surfactants to prevent affects on environmentally sensitive areas, habitat, native vegetation, landscape plantings, agricultural crops, adjacent residential, commercial or recreation areas, streams, or water bodies.

Before specifying preemergents, the District Landscape Architect and District Landscape Specialist should be consulted to determine the possibility of future planting.

Topic 707 – Slope Treatment Under Structures

707.1 Policy

Structure end slope should be treated to:

(a) Protect slopes from erosion.

(b) Improve aesthetics.

(c) Reduce long term maintenance costs.

Caltrans maintenance, landscape architecture, materials, design, and other affected units will furnish input to determine slope treatment needed at each site. Local agency input should be obtained for urban undercrossings.

All types of slope treatments require adequate drainage facilities for water from the upper roadway. Inadequate drainage is a major source of slope erosion.

707.2 Guidelines for Slope Treatment

(a) Full slope paving shall be installed where it is anticipated that erosion by pedestrians, wind, storm water, or other causes will occur. High landscape maintenance costs caused by inadequate moisture, sunlight, instability to establish vegetation etc., may also justify the use of full slope paving in lieu of planting. The District Landscape Architect will provide aesthetic input slope paving and identify irrigation conduit location(s).

(b) Landscaped structure end slopes may be justified when adjacent slopes are landscaped and when landscaping is compatible with adjacent development. Conditions must exist where plants would have a strong likelihood of survival.

(c) Bare slopes have minimum initial costs and higher maintenance costs which vary with the site. Bare structure end slopes may be justified at rural sites and other areas where anticipated maintenance activity will be low and there is little likelihood for erosion. Appropriate drainage design is critical when slopes are left bare.

(d) Adequate drainage facilities must be provided to prevent saturation of abutment foundation materials and damage to slope treatment.

(e) Additional protection may be required at stream crossings to provide for flow velocity.

707.3 Procedure

Based on consultation with the District Landscape Architect and Structures Bridge Architect and in consideration of economic and aesthetic factors, the District will determine, and set forth with the bridge site plan submittal, the type of slope treatment indicating whether:

(a) The Division of Engineering Services is to design the slope treatment with the bridge and include the cost in the Structure items; or

(b) The District will design the slope treatment and include the details with the road plans.

CHAPTER 850 – PHYSICAL STANDARDS

Topic 851 – General

Index 851.1 – Introduction

This chapter deals with the selection of drainage facility material type and sizes including pipes, pipe liners, pipe linings, drainage inlets and trench drains.

851.2 Selection of Material and Type

The choice of drainage facility material type and size is based on the following factors:

(1) Physical and Structural Factors. Of the many physical and structural considerations, some of the most important are:

(a) Durability.

(b) Headroom.

(c) Earth Loads.

(d) Bedding Conditions.

(e) Conduit Rigidity.

(f) Impact.

(g) Leak Resistance.

(2) Hydraulic Factors. Hydraulic considerations involve:

(a) Design Discharge.

(b) Shape, slope and cross sectional area of channel.

(c) Velocity of approach.

(d) Outlet velocity.

(e) Total available head.

(f) Bedload.

(g) Inlet and outlet conditions.

(h) Slope.

(i) Smoothness of conduit.

(j) Length.

Suggested values for Manning's Roughness coefficient (n) for design purposes are given in Table 851.2 for each type of conduit. See Index 866.3 for use of Manning's formula.

Table 851.2

Manning "n" Value for Alternative Pipe Materials[(1)]

Type of Conduit		Recommended Design Value	"n" Value Range
Corrugated Metal Pipe (2)			
(Annular and Helical) (3)			
2⅔" x ½"	corrugation	0.025	0.022 - 0.027
3" x 1"	"	0.028	0.027 - 0.028
5" x 1"	"	0.026	0.025 - 0.026
6" x 2"	"	0.035	0.033 - 0.035
9" x 2½"	"	0.035	0.033 - 0.037
Concrete Pipe			
Pre-cast		0.012	0.011 - 0.017
Cast-in-place		0.013	0.012 - 0.017
Concrete Box		0.013	0.012 - 0.018
Plastic Pipe			
Smooth Interior		0.012	0.010 - 0.013
Corrugated Interior		0.022	0.020 - 0.025
Spiral Rib Metal Pipe			
¾" (W) x 1" (D) @ 11½" o/c		0.013	0.011 - 0.015
¾" (W) x ¾" (D) @ 7½" o/c		0.013	0.012 - 0.015
¾" (W) x 1" (D) @ 8½" o/c		0.013	0.012 - 0.015
Composite Steel Spiral Rib Pipe		0.012	0.011 - 0.015
Steel Pipe, Ungalvanized		0.015	--
Cast Iron Pipe		0.015	--
Clay Sewer Pipe		0.013	--
Polymer Concrete Grated Line Drain		0.011	0.010 - 0.013

Notes:

(1) Tabulated n-values apply to circular pipes flowing full except for the grated line drain. See Note 5.

(2) For lined corrugated metal pipe, a composite roughness coefficient may be computed using the procedures outlined in the HDS No. 5, Hydraulic Design of Highway Culverts.

(3) Lower n-values may be possible for helical pipe under specific flow conditions (refer to FHWA's publication Hydraulic Flow Resistance Factors for Corrugated Metal Conduits), but in general, it is recommended that the tabulated n-value be used for both annular and helical corrugated pipes.

(4) For culverts operating under inlet control, barrel roughness does not impact the headwater. For culverts operating under outlet control barrel roughness is a significant factor. See Index 825.2 Culvert Flow.

(5) Grated Line Drain details are shown in Standard Plan D98G-D98J and described under Index 837.2(6) Grated Line Drains. This type of inlet can be used as an alternative at the locations described under Index 837.2(5) Slotted Drains. The carrying capacity is less than 18-inch slotted (pipe) drains.

(6) *Spiral Rib Aluminum.* Aluminum spiral rib pipe is fabricated using sheet aluminum and continuous helical lock seam fabrication as used for helical corrugated metal pipe. The manufacturing complies with Section 66, “Corrugated Metal Pipe,” of the Standard Specifications, except for profile and fabrication requirements. Aluminum spiral rib pipe is fabricated with either: three rectangular ribs spaced midway between seams with ribs 3/4" wide x 3/4" high at a maximum rib pitch of 7-1/2 inches or two rectangular ribs and one half-circle rib equally spaced between seams with ribs 3/4" wide x 1" high at a maximum rib pitch of 11-1/2 inches with the half-circle rib diameter spaced midway between the rectangular ribs. Figure 855.3A should be used to determine the limitations on the use of spiral rib aluminum pipe for the various levels of pH and minimum resistivity.

852.5 Structural Metal Plate

(1) *Pipe and Arches*. Structural plate pipes and arches are available in steel and aluminum for the diameters and thickness as shown on Tables 856.3M, N, O & P.

(2) *Strength Requirements*.

(a) Design Standards.

- Corrugation Profiles – Structural plate pipe and arches are available in a 6" x 2" corrugation for steel and a 9" x 2½" corrugation profile for aluminum.
- Metal Thickness – structural plate pipe and pipe arches are available in thickness as indicated on Tables 856.3M, N, O & P.
- Height of Fill – The allowable height of cover over structural plate pipe and pipe arches for the available diameters and thickness are shown on Tables 856.3M, N, O & P.

Where a maximum overfill is not listed on these tables, the pipe or arch size is not normally available in that thickness. All pipe sections provided in Table 856.3 conform to handling and installation flexibility requirements of AASHTO LRFD. Strutting of culverts, as depicted on Standard Plan D88A, is typically necessary if the pipe is used as a vertical shaft or if the backfill around the pipe is being removed in an unbalanced manner.

(b) Basic Premise. To properly use the above mentioned tables, the designer should be aware of the premises on which the tables are based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications, the conditions of cover, and pipe or arch size required by the plans and the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.

(c) Limitations. In using the tables, the following restrictions should be kept in mind.

- The values given for each size of structural plate pipe or arch constitute the maximum height of overfill or cover over the pipe or arch for the thickness of metal and kind of corrugation.
- The thickness shown is the structural minimum. For steel pipe or pipe arches, where abrasive conditions are anticipated, additional metal thickness for the invert plate(s) or a paved invert should be provided when required to fulfill the design service life requirements. Table 855.2C may be used. See Index 855.2 Abrasion and Tables 855.2A, 855.2D and 855.2F.

- Where needed, adequate provisions for corrosion resistance must be made to achieve the required design service life called for in the references mentioned herein.

(d) Tables 856.3M & P show the limit of heights of cover for structural plate arches based on the supporting soil sustaining a bearing pressure of 3 tons per square foot at the corners. Special Designs. If the height of overfill exceeds the tabular values, or if the foundation investigation reveals that the supporting soil will not develop the bearing pressure on which the overfill heights for structural plate pipe or pipe arches are based, a special design prepared by DES - Structures Design is required.

(3) Arches. Design details with maximum allowable overfills for structural plate arches, with cast in place concrete footings may be obtained from DES - Structures Design.

(4) Vehicular Underpasses. Design details with maximum allowable overfills for structural plate vehicular underpasses with spans from 12 feet 2 inches to 20 feet 4 inches, inclusive, are given in the Standard Plans. These designs are based on “factored” bearing soil pressures from 2.5 tons per square foot to 11 tons per square foot.

(5) Special Shapes.

(a) Long Span.

- Arch
- Low Profile Arch
- High Profile Arch

(b) Ellipse. (Text Later)

- Vertical
- Horizontal

(6) Tunnel Liner Plate. The primary applications for tunnel liner plate include lining large structures in need of a structural repair, or culvert installations through an existing embankment that can be constructed by conventional tunnel methods. Typically, tunnel liner plate is not used for direct burial applications where structural metal plate pipe is recommended. DES - Structures Design will prepare designs upon request. See Index 853.7 for structural repairs.

852.6 Plastic Pipe

Plastic pipe is a generic term which currently includes three independent materials; the Standard Specifications states plastic pipe shall be made of either high density polyethylene (HDPE), polyvinyl chloride (PVC), or polypropylene (PP) material. See Index 852.6(2)(a) Strength Requirements for allowed materials and wall profile types.

(1) Durability. Caltrans standards regarding the durability of plastic pipe are based on the long term performance of its material properties. Each of the three forms of plastic pipe culverts (HDPE, PVC, and PP) exhibit good abrasion resistance and are virtually corrosion free. See Index 855.2 Abrasion and Index 855.5 Material Susceptibility to Fire. Also, see Tables 855.2A, 855.2E and 855.2F. The primary environmental factor currently considered in limiting service life of plastic materials is ultraviolet (UV) radiation, typically from sunlight exposure. While virtually all plastic pipes contain some amount of UV protection, the level of protection is not equal. Polyvinyl chloride resins used for pipe rarely incorporate UV protection (typically Titanium Dioxide) in amounts adequate to offset long term exposure to direct sunlight. Therefore, frequent exposure (e.g., cross culverts with exposed ends) can

lead to brittleness and such situations should be avoided. Conversely, testing performed to date on HDPE and PP products conforming to specification requirements for inclusion of carbon black have exhibited adequate UV resistance. PVC and PP pipe exposed to freezing conditions can also experience brittleness and such situations should be avoided if there is potential for impact loadings, such as maintenance equipment or heavy (3" or larger) bedload during periods of freeze. Plastic pipes can also fail from long term stress that leads to crack growth and from chemical degradation. Improvements in plastic resin specifications and testing requirements has led to increased resistance to slow crack growth. Inclusion of anti-oxidants in the material formulation is the most common form of delaying the onset of chemical degradation, but more thorough testing and assessment protocols need to be developed to more accurately estimate long term performance characteristics and durability.

(2) *Strength Requirements.*

(a) Design Standards

- Materials - Plastic pipe shall be either Type C (corrugated exterior and interior) corrugated polyethylene pipe, Type S (corrugated exterior and smooth interior) corrugated polyethylene pipe, corrugated polyvinyl chloride pipe, or dual wall polypropylene pipe (corrugated exterior and smooth interior).
- Height of Fill - The allowable overfill heights for plastic pipe for various diameters are shown in Tables 856.4 and 856.5.

852.7 Special Purpose Types

(1) *Smooth Steel.* Smooth steel (welded) pipe can be utilized for drainage facilities under conditions where corrugated metal or concrete pipe will not meet the structural or design service life requirements, or for certain jacked pipe operations (e.g., auger boring).

(2) *Composite Steel Spiral Rib Pipe.* Composite steel spiral rib pipe is a smooth interior pipe with efficient hydraulic characteristics. See Table 851.2.

Composite steel spiral rib pipe with its interior polyethylene liner exhibits good abrasion resistance and also resists waterside corrosion found in a typical stormdrain or culvert environment. The exterior of the pipe is protected with a polyethylene film, which offers resistance to corrosive backfills. The pipe will meet a 50-year maintenance-free service life under most conditions. See Table 856.3G for allowable height of cover.

(3) *Proprietary Pipe.* See Index 110.10 for further discussion and guidelines on the use of proprietary items.

Topic 853 – Pipe Liners and Linings for Culvert Rehabilitation

853.1 General

This topic discusses alternative pipe liner and pipe lining materials specifically intended for culvert repair and does not include materials used for Trenchless Excavation Construction (e.g., pipe jacking, pipe ramming, augur boring), joint repair, various types of grouting, or standard pipe materials that are presented elsewhere in Chapter 850 and in the Standard Plans and Standard Specifications.

Many new products and techniques have been developed that often make complete replacement with open cut as shown in the Standard Plans unnecessary. When used appropriately, these new products and techniques can benefit the Department in terms of increased mobility, cost, and safety to both the public and contractors. Design Information Bulletin 83 (DIB 83) outlines a collection of procedures that are cost-effective for their location and that will meet the needs of their particular area, supplementing Topic 853. Use the following link: https://dot.ca.gov/-/media/dot-media/programs/design/documents/dib83-04-a11y.pdf for further information.

853.2 Caltrans Host Pipe Structural Philosophy

In general, if the host (i.e., existing) pipe cannot be made capable of sustaining design loads, it should be replaced rather than rehabilitated. This is a conservative approach and when followed eliminates the need to make a detailed evaluation of the liner's ability to effectively accept and support dead and live loads. Prior to making the decision whether or not to rehabilitate the culvert and/or which method to choose, a determination of the structural integrity of the host pipe must be made. If rehabilitation of the culvert is determined to be a feasible option, existing voids within the culvert backfill or in the base material under the existing culvert identified either by Maintenance (typically as part of their culvert management system) or already noted in the Geotechnical Design Report, should be filled with grout to re-establish its load carrying capability. Therefore, structural considerations for pipe liners are generally limited to their ability to withstand construction handling and/or grouting pressures. When a structural repair is needed, contact Underground Structures within DES – Structures Design. See Index 853.7.

853.3 Problem Identification and Coordination

Before various alternatives for liners or linings can be selected, the first step following a site investigation which may include taking soil and water samples and pipe wall thickness measurements, is to determine the actual cause of the problem. Relative to Caltrans host pipe structural philosophy, the host pipe may be in need of stabilization, rehabilitation or replacement. Further, it will need to be determined if the structure is at the end of its maintenance-free service life, whether it has been damaged by mechanical abrasion, or corrosion (or both) and if there are any changes to the hydrology or habitat (e.g. fish passage). To make these determinations, the Project Engineer should coordinate with the District Maintenance Culvert Inspection team, Hydraulics and Environmental units. Further assistance may be needed from Geotechnical Design, the Corrosion Technology Branch within DES, Underground Structures and/or Structures Maintenance within DES. Prior to a comprehensive inspection either by trained personnel or camera, it may also be necessary to first clean out the culvert. Problem identification and assessment, and coordination with Headquarters and DES, is discussed in greater detail in DIB 83. Use the following link; https://dot.ca.gov/-/media/dot-media/programs/design/documents/dib83-04-a11y.pdf.

853.4 Alternative Pipe Liner Materials

Similar to the basic policy in Topic 857.1 for alternative pipes, when two or more liner materials meet the design service life and minimum thickness requirements for various materials that are outlined under Topic 855, as well as hydraulic requirements, the plans and specifications should provide for alternative pipe liners to allow for optional selection by the contractor. A table of allowable alternative pipe liner materials for culverts and drainage systems is included as Table 853.1A. This table also identifies the various diameter range limitations and whether annular space grouting is needed. Sliplining consists of sliding a new culvert inside an existing distressed culvert as an alternative to total replacement. See DIB No 83; https://dot.ca.gov/-/media/dot-media/programs/design/documents/dib83-04-a11y.pdf.

The plastic pipeliners listed in the notes under Table 853.1A are installed as slipliners, however, other standard pipe types that are described in Topic 852 (e.g., metal), may be equally viable as material options to be added as sliplining alternatives.

Table 853.1A

Allowable Alternative Pipe Liner Materials

Allowable Alternatives	Diameter Range [(1)]	Annular Space Grouting
Plastic Pipe [(2)]	15" – 120"	Yes
CIPP	8" – 96"	No
MSWPVCPLED	6" – 30"	No
SWPVCPLFD	21" – 108"	Yes

Abbreviations:

CIPP – Cured in Place Pipe

SWPVCPLFD – Spiral Wound PVC Pipe Liner (Fixed Diameter)

MSWPVCPLED – Machine Spiral Wound PVC Pipe Liner (Expandable Diameter)

Note:

[(1)] Headquarters approval needed for pipe liner diameters 60 inches or larger. Diameter range represents liners only, not Caltrans standard pipe.

[(2)] The designer must edit the following plastic pipeliner list within SSP 71-3.07, Plastic Pipeliners, to suit the work:

- Type S corrugated high density polyethylene (HDPE) and polypropylene (PP) pipes conforming to the provisions in Section 64, “Plastic Pipe,” of the Standard Specifications; or
- Standard Dimension Ratio (SDR) 35 polyvinyl chloride (PVC) pipe conforming to the requirements in AASHTO Designation: M 278 and ASTM Designation: F 679; or
- Polyvinyl chloride (PVC) closed profile wall pipe conforming to the requirements in ASTM Designation: F 1803, F 794 (Series 46); or
- Polyvinyl chloride (PVC) dual wall corrugated pipe conforming to the requirements in ASTM Designation: F 794 (Series 46), and ASTM Designation F 949; or
- Polypropylene (PP) dual wall corrugated pipe conforming to the requirements in ASTM Designation: F2881 and AASHTO Designation: M 330; or
- High density polyethylene (HDPE) solid wall pipe conforming to the requirements in AASHTO M 326 and ASTM Designation: F 714; or
- Large diameter high density polyethylene (HDPE) closed profile wall pipe conforming to the requirements in ASTM Designation: F 894.

Table 853.1B provides a guide for plastic pipeliner selection in abrasive conditions to achieve a 50-year maintenance-free service life.

For further information on sliplining using plastic pipe liners including available dimensions and stiffness, see DIB 83. Use the following link: https://dot.ca.gov/-/media/dot-media/programs/design/documents/dib83-04-a11y.pdf.

853.5 Cementitious Pipe Lining

This method may be used to line corroded corrugated steel pipes ranging from 12 inches to a maximum of 36 inches diameter and involves lining an existing culvert with concrete, shotcrete or mortar using a lining machine. If the bedload is abrasive, alternative cementitious materials such as calcium aluminate mortar or geopolymer mortar may be selected from the Authorized Materials list for cementitious pipeliners. See Table 855.2F and Section 71-3.10, Cementitious Pipeliners, of the Standard Specifications for specifications. Regardless of type of cementitious material used, the resulting lining is a minimum of one inch thick when measured over the top of corrugation crests and has a smooth surface texture. As with other liners, the pipes must first be thoroughly cleaned and dried. For diameters between 12 and 24 inches, the cement mortar is applied by robot. The mortar is pumped to a head, which rotates at high speed using centrifugal force to place the mortar on the walls. A conical-shaped trowel attached to the end of the machine is used to smooth the walls. The maximum recommended length of small-diameter pipe that can be lined using this method is approximately 650 feet. Although this method will line larger diameter pipes, it is mostly appropriate for non-human entry pipes (less than 30 inches). Generally, most problems with steel pipe are limited to the lower 180 degrees, therefore, in larger diameter metal pipes where human entry is possible, invert paving may be all that is required. See Index 853.6.

853.6 Invert Paving with Concrete

(1) *Existing Corrugated Metal Pipe (CMP).* One of the most effective ways to rehabilitate corroded and severely deteriorated inverts of CMP that are large enough for human entry (with equipment) is by paving them with reinforced concrete shotcrete or authorized cementitious material. Standard Specification Section 15-6.04 includes specifications for preparing the surface of the culvert invert, installing bar reinforcement and anchorage devices, and paving the invert with concrete, shotcrete or authorized cementitious material. For most non-abrasive sites, concrete may comply with the requirements for minor concrete or shotcrete. See index 110.12 Tunnel Safety Orders. Generally, this method is feasible for pipes 48 inches in diameter and larger. If abrasion is present, see Table 855.2F for minimum

Table 853.1B

Guide for Plastic Pipeliner Selection in Abrasive Conditions[2] to Achieve 50 Years of Maintenance-Free Service Life

		Abrasion Level[1]		
MATERIAL		4	5	6
Standard Dimension Ratio (SDR) 35 PVC [3]	(46 psi)	4" – 48"	12"- 48"	36"– 48"
	(75 psi)	18" – 48"	18" – 48"	30" – 48"
	(115 psi)	18" – 48"	18" – 48"	27" – 48"
Standard Dimension Ratio (SDR) PVC[4] (AWWA C900 & C905)	SDR 41	30" – 36"	30" – 36"	-
	SDR 32.5	30" – 36"	30" – 36"	30" – 36"
	SDR 25	4" – 36"	8" – 36"	24" – 36"
	SDR 21	14" – 24"	14" – 24"	20" – 24"
	SDR 18	4" – 24"	6" – 24"	18" – 24"
	SDR 14	4" – 12"	4" – 12"	-
PVC closed profile wall (ASTM F 1803)		18" – 60"	42"– 60"	-
Corrugated PVC (ASTM F 794 & F 949)	(46 psi)	18" – 36"	-	-
	(115 psi)	15"	-	-
Standard Dimension Ratio (SDR) HDPE[3] (AASHTO M 326 and ASTM Designation F 714)	SDR 41	10" – 63"	36" – 63"	-
	SDR 32.5	8" – 63"	30" – 63"	-
	SDR 26	6" – 63"	24" – 63"	-
	SDR 21	5" – 63"	20" – 63"	54" – 63"
	SDR 17	5" – 55"	16" – 55"	42" – 55"
	SDR 15.5	5" – 48"	14" – 48"	42" – 48"
	SDR 13.5	5" – 42"	12" – 42"	34" – 42"
	SDR 11	5" – 36"	10" – 36"	28" – 36"
	SDR 9	5" – 24"	8" – 24"	22"
Polyethylene (PE) large diameter profile wall sewer and drain pipe (ASTM F 894)	RSC[5] 160	18" – 120"	120"	-
	RSC[5] 250	33" – 108"	96" – 108"	-

NOTES:

(1) See Tables 855.2A and 855.2F for Abrasion Level Descriptions and minimum thickness.

(2) No restrictions for Abrasion Levels 1 through 3.

(3) Measured pipe designated SDR is measured to outside diameter.

(4) Measured to inside diameter.

(5) RSC = Ring Stiffness Class

material thickness of concrete or authorized material. Concrete should have a minimum compressive strength of 6,000 psi at 28 days and the aggregate source should be harder material than the streambed load and have a high durability index (consult with District Materials Branch for sampling and recommendation). The maximum grading specified (1.5 inch) for coarse aggregate may need to be modified if the concrete must be pumped. The abrasion resistance of cementitious materials is affected by both its compressive strength and hardness of the aggregate. There is a correlation between decreasing the water/cement ratio, increasing compressive strength and increasing abrasion resistance. Therefore, where abrasion is a significant factor, the lowest practicable water/cement ratios and the hardest available aggregates should be used.

Paving thickness will range from 2 inches to 13 inches depending on abrasiveness of site based on Table 855.2A, and paving limits typically vary from 90 to 120 degrees for the internal angle. See Index 855.2 and Table 855.2F. Note that in Table 855.2F cementitious concrete is not recommended for extremely abrasive conditions (Level 6 in Table 855.2A). For extremely abrasive conditions alternative materials are recommended such as abrasion resistant concrete (calcium aluminate), steel plate or adding RSP. Calcium aluminate abrasion resistant concrete or mortar may be selected from the Authorized Materials list for concrete invert paving. If hydraulically feasible, a flattened invert design may be warranted.

Consult the District Hydraulic Branch for a recommendation.

Where there is significant loss of the pipe invert, it may be necessary to tie the concrete to more structurally sound portions of the pipe wall in order to transfer compressive thrust of culvert walls into the invert slab to create a “mechanical” connection using welding studs, angle iron or by other means. When a mechanical connection is used, paving limits may vary up to 180 degrees for the internal angle. These types of repairs should be treated as a special design and consultation with the Headquarters Office of Highway Drainage Design within the Division of Design and the Underground Structures unit of Structures Design within the Division of Engineering Services (DES) is advised. Depending on the size of the culvert being paved, pipes with significant invert loss often also have a significant loss of structural backfill with voids present. Where large voids are present, consultation with Geotechnical Services within the Division of Engineering Services (DES) is advised to develop a grouting plan.

See DIB 83 for some invert paving case studies using the following link: http://www.dot.ca.gov/hq/oppd/dib/dib83-01-12.htm#h

(2) *Existing RCB and RCP*. For existing reinforced concrete boxes (RCB) and reinforced concrete pipes (RCP) with worn inverts and exposed reinforcing steel (generally from abrasive bedloads), the same paving thickness considerations outlined under Index 853.6(1) will apply. However, depending on the structural condition, the existing steel reinforcement may need to be augmented. Consultation with Structures Maintenance and Underground Structures within DES is recommended.

(3) *Existing Plastic Pipe.* Generally, concrete invert paving is not feasible for plastic pipes because the cement will not adhere to plastic. However, it may be possible to create a “mechanical” connection by other means but these types of repairs should be treated as a special design and consultation with the Headquarters Office of Highway Drainage Design within the Division of Design and the Underground Structures unit of Structures Design within the Division of Engineering Services (DES) is advised.

Estimating Years to Perforation of Steel Culverts," is part of a Standard California Department of Transportation Test Method derived from highway culvert investigations. This chart alone is not used for determining service life because it does not consider the effects of abrasion or overfill; it is for estimating the years to the first corrosion perforation of the wall or invert of the CSP. Additional gauge thickness or invert protection may be needed if the thickness for structural requirements (i.e., for overfill) is inadequate for abrasion potential.

Table 855.2E indicates relative abrasion resistance properties of pipe and lining materials and summarizes the findings from "Evaluations of Abrasion Resistance of Pipe and Pipe Lining Materials Final Report FHWA /CA/TL-CA01-0173 (2007)". This report may be viewed at the following web address: https://rosap.ntl.bts.gov/view/dot/27517. See Figure 855.2.

Figure 855.2

Abrasion Test Panels

Various culvert material test panels shown in Figure 855.2 after 1 year of wear at site with moderate to severe abrasion (velocities generally exceed 13 ft/s with heavy bedload). The report included HDPE and PVC plastic pipe materials, but not PP. Additional studies have shown that PP abrasion resistance could exceed that of HDPE, however industry recommends using the abrasion values assigned to corrugated HDPE for PP pipe until specific abrasion resistance data can be obtained.

Table 855.2F is based on Tables 855.2D and 855.2E and constitutes a guide for selecting the minimum material thickness of abrasive resistant invert protection for various materials to achieve 50 years of maintenance-free service life.

Structural metal plate pipe and arches provide a viable option for large diameter pipes (60 inches or larger) in abrasive environments because increased thickness can be specified for the lower 90 degrees or invert plates. If the thickness for structural requirements is inadequate for abrasion potential, it is recommended to apply the increased thickness to the lower 90 degrees of the pipe only. Arches, which have a relatively larger invert area than circular pipe, generally will provide a lower abrasion potential from bedload being less concentrated.

Table 855.2A

Abrasion Levels and Materials

Abrasion Level	General Site Characteristics	Allowable Pipe Materials and Lining Alternatives
Level 1	• Bedloads of silts and clays or clear water with virtually no abrasive bed load. No velocity limitation.	All pipe materials listed in Table 857.2 allowable for this level. No abrasive resistant protective coatings listed in Table 855.2C needed for metal pipe.
Level 2	• Moderate bed loads of sand or gravel • Velocities ≥ 1 ft/s and ≤ 5 ft/s (See Note 1)	All allowable pipe materials listed in Table 857.2 with the following considerations: • Generally, no abrasive resistant protective coatings needed for steel pipe. • Polymeric, or bituminous coating or an additional gauge thickness of metal pipe may be specified if existing pipes in the same vicinity have demonstrated susceptibility to abrasion and thickness for structural requirements is inadequate for abrasion potential.
Level 3	• Moderate bed load volumes of sands, gravels and small cobbles. • Velocities > 5 ft/s and ≤ 8 ft/s (See Note 1)	All allowable pipe materials listed in Table 857.2 with the following considerations: • Steel pipe may need one of the abrasive resistant protective coatings listed in Table 855.2C or additional gauge thickness if existing pipes in the same vicinity have demonstrated susceptibility to abrasion and thickness for structural requirements is inadequate for abrasion potential. • Aluminum pipe may require additional gauge thickness for abrasion if thickness for structural requirements is inadequate for abrasion potential. • Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (equivalent to galv. Steel) where pH < 6.5 and resistivity < 20,000. Lining alternatives: • PVC, • Corrugated or Solid Wall HDPE, • Dual Wall PP, • CIPP

Note:

(1) If bed load volumes are minimal, a 50% increase in velocity is permitted.

Table 855.2A

Abrasion Levels and Materials (Cont.)

Abrasion Level	General Site Characteristics	Allowable Pipe Materials and Lining Alternatives
Level 4	• Moderate bed load volumes of angular sands, gravels, and/or small cobbles/rocks. (See Note 1) • Velocities > 8 ft/s and ≦ 12 ft/s	All allowable pipe materials listed in Table 857.2 with the following considerations: • Steel pipe will typically need one of the abrasive resistant protective coatings listed in Table 855.2C or may need additional gauge thickness if thickness for structural requirements is inadequate for abrasion potential. • Aluminum pipe not recommended. • Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (wear rate equivalent to galv. steel) where pH < 6.5 and resistivity < 20,000 if thickness for structural requirements is inadequate for abrasion potential. • Increase concrete cover over reinforcing steel for RCB (invert only). RCP generally not recommended. • Corrugated HDPE (Type S) limited to ≥ 48" min. diameter. Corrugated HDPE Type C not recommended. • Dual Wall PP • Corrugated PVC limited to ≥ 18" min. diameter Lining alternatives: • Closed profile or SDR 35 PVC (corrugated and ribbed PVC limited to ≥ 18" min. diameter. • SDR HDPE • CIPP (min. thickness for abrasion specified) • Concrete and authorized cementitious pipeliners and invert paving. See Table 855.2F.

Note:

(1) For minor bed load volumes, use Level 3.

Table 855.2A

Abrasion Levels and Materials (Cont.)

Abrasion Level	General Site Characteristics	Allowable Pipe Materials and Lining Alternatives
Level 5	• Moderate bed load volumes of angular sands and gravel or rock (See Note 1). • Velocities > 12 ft/s and ≤ 15 ft/s	• Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (wear rate equivalent to galv. steel) where pH < 6.5 and resistivity < 20,000 if thickness for structural requirements is inadequate for abrasion potential. • For steel pipe invert lining additional gauge thickness is recommended if thickness for structural requirements is inadequate for abrasion potential. See lining alternatives below. • Increase concrete cover over reinforcing steel for RCB (invert only). RCP generally not recommended Lining alternatives: • Closed profile (≥ 42 in) or SDR 35 PVC (PVC liners not recommended when freezing conditions are often encountered and cobbles or rocks are present) • SDR HDPE • CIPP (with min. thickness for abrasion specified) • Concrete and authorized cementitious pipeliners and invert paving. See Table 855.2F.

Note:

(1) For minor bed load volumes, use Level 3.

Table 855.2F

Guide for Minimum Material Thickness of Abrasive Resistant Invert Protection to Achieve 50 Years of Maintenance-Free Service Life

Abrasion Level & Flow Velocity (ft/s)	Channel Materials	Concrete [4] (in)	Steel Pipe & Plate (in)	Aluminum Pipe & Plate (in)	PVC (in)	HDPE (in)	PP (in)	CIPP (in)	Calcium Aluminate Abrasion Resistant Concrete [5] (in)	Mortar [5]	
										Calcium Aluminate (in)	Geopolymer (in)
Level 4 > 8 – ≤ 12	Abrasive	2 – 4	0.052	0.075 – 0.164	0.1	0.125 – 0.25	0.125 – 0.25	0.1 – 0.3	(6)	1-2	2-4
Level 5 > 12 – ≤ 15	Abrasive	4 – 13	0.052 – 0.18	(2)	0.1 – 0.35	0.25 – 0.875	0.25 – 0.875	0.3 – 0.70	3[6]	2-5	4-13
Level 6 > 12 – ≤ 20	Abrasive & Heavy bedloads	(1)	0.109 – 0.5	(2)	0.25 – 1.0[3]	0.625 – 2.5	0.625 – 2.5[3]	0.5 – 2	3 – 5	5-8	(1)

Notes:

(1) For flow velocity > 12 ft/s ≤ 14 ft/s use 9" – 15". For > 14 ft/s use CRSP or other abrasion resistant layer special design with, or in lieu of concrete or geopolymer mortar.

(2) Not recommended without invert protection.

(3) PVC and PP liners not recommended when freezing conditions are often encountered and cobbles and rocks are present.

(4) Values shown based on RCP abrasion test results. See Table 855.2E. Results may differ from concrete specified under 71-3.04 for invert paving which must have a minimum compressive strength of 6,000 psi at 28 days and 1 ½-inch maximum grading.

(5) See Authorized Materials List for Cementitious Pipeliners and Concrete Invert Paving: https://dot.ca.gov/programs/engineering-services/authorized-materials-lists. Standard Mortar (Section 51-1.02F of the Standard Specifications) not recommended for Abrasion Level 4 or higher.

(6) Minimum thickness recommended is 3". Not practical or economically viable for Level 4. Consider calcium aluminate mortar or standard concrete (Section 90 of the Standard Specifications) for lower range of Level 5.

culverts smaller than 30 inches or larger diameters with insignificant abrasive bedload volumes).

Abrasion resistance for any concrete lining is dependent upon the thickness, quality, strength, and hardness of the aggregate and compressive strength of the concrete as well as the velocity of the water flow coupled with abrasive sediment content and acidity. Abrasion resistant concrete or mortar made from calcium aluminate provides much improved abrasion resistance over cementitious concrete and should be considered as a viable countermeasure in extremely abrasive conditions (i.e., velocity greater than 15 feet per second with heavy bedload). See Table 855.2F.

Plastic materials typically exhibit good abrasion resistance but service life is constrained by the manufactured thickness of typical pipe profiles. PP, PVC, and HDPE corrugated pipe are limited for their use in moderate and heavy bedload abrasion conditions by the combined manufactured inner liner and corrugated wall thicknesses. For culvert rehabilitation, PVC and HDPE pipe slip lining products (e.g. solid wall HDPE) are viable options for applications in moderate and heavy bedload abrasion conditions (see Table 855.2A).

Table 855.2A can be used as a "preliminary estimator" of abrasion potential for material selection to achieve the required service life, however, it incorporates only three of the primary abrasion factors; bedload volume, bedload type and flow velocity and the general assumption is the materials are angular, hard and abrasive. As discussed above, the other factors that are not used in the table should also be carefully considered. For example, under similar hydraulic conditions, heavy volumes of hard, angular sand may be more abrasive than small volumes of relatively soft, large or rounded rocks. Furthermore, two sites with similar site characteristics, but different hydrologic characteristics, i.e., volume, duration and frequency of stream flow in the culvert, will likely also have different abrasion levels. Table 855.2B can be used as a guide with Table 855.2A to determine the maximum size of material that can be moved through a pipe. Field observations of channel bed material both upstream and downstream from the pipe are extremely important for estimating the size range of transportable material in the channel.

855.3 Corrosion

Corrosion is the destructive attack on a pipe by a chemical reaction with the materials surrounding the pipe. Corrosion problems can occur when metal pipes are used in locations where the surrounding materials have excess acidity or alkalinity. The relative acidity of a substance is often expressed by its pH value. The pH scale ranges from 1 to 14, with 1 representing extreme acidity, and 14 representing extreme alkalinity, and 7 representing a neutral substance. The closer the pH value is to 7, the less potential the substance has for causing corrosion.

Corrosion is an electrolytic process and requires an electrolyte (generally moisture) and oxygen to proceed. As a result, it has the greatest potential for causing damage in soils that have a relative high ability to pass electric current. The ability of a soil to convey current is expressed as its resistivity in ohm-cm, and a soil with a low resistivity has a greater ability to conduct electricity. Very dry areas (e.g., desert environments) have a limited availability of electrolyte, and totally and continuously submerged pipes have limited oxygen availability. These extreme conditions (among others) are not well represented by AltPipe, and some adjustment in the estimated service life for pipes in these conditions should be made. See Index 857.2

Table 855.4B

Guide for Minimum Cover Requirements for Cast-In-Place and Precast Reinforced Concrete Structures[3] for 50-Year Design Life in Chloride Environments

Chloride Concentration (ppm)			
500 to 2000	2001 to 5000	5001 to 10000	10000 +
1.5 in.[1]	2.5 in.[1]	3 in.[1]	4 in.[1]
1.5 in.[2]	1.5 in.[2]	2 in.[2]	3 in.[2]

Notes:

[1] Supplementary cementitious materials are required. Typical minimum requirement consists of 675#/cy minimum cementitious material with 75% by weight of Type II or Type V portland cement and 25% by weight of either fly ash or natural pozzolan. A maximum w/cm ratio of 0.40 is specified. Fly ash or natural pozzolan may have a CaO content of up to 10%. Section 90-1.02B(3) of the Standard Specifications provides requirements.

[2] Additional supplementary cementitious materials per the requirements of Section 90-1.02B(3) of the Standard Specifications are required in order to achieve the listed reduction in concrete cover.

[3] Does not include RCP.

restrictions for various ranges of sulfate concentrations in soil and water for all cast in place and precast construction of drainage structures.

For pH ranging between 7.0 and 3.0 and for sulfate concentrations between 1500 and 15,000 ppm, concrete mix designs conforming to the recommendations given in Table 855.4A should be followed. Higher sulfate concentrations or lower pH values may preclude the use of concrete or would require the designer to develop and specify the application of a complete physical barrier. Reinforcing steel can be expected to respond to corrosive environments similarly to the steel in CSP.

Table 855.4B provides a guide for minimum concrete cover requirements for various ranges of chloride concentrations in soil and water for all precast and cast in place construction of drainage structures.

(1) RCP. In relatively severe acidic, chloride or sulfate environments (either in the soil or water) as identified in the project Materials Report, the means for offsetting the effects of the corrosive elements is to either increase the cover over the reinforcing steel, increase the cementitious material content, or reduce the water/ cementitious material ratio. The identified constituent concentration levels should be entered into AltPipe to verify what combinations of increased cover (in 1/4-inch intervals from 1 inch to a maximum of 1-1/2 inches), increased cementitious material content (in increments of 47 pounds from 470 pounds to a maximum of 564 pounds), will provide the necessary service life (typically 50 years). Per an agreement with Industry, the water to cementitious material ratio is set at 0.40. AltPipe is specifically programmed to provide RCP mix and cover designs that are compatible with industry practice, and are based on their agreements with Caltrans. For corrosive condition installations such as low pH (<4.5), Chlorides (>2,000 ppm) or Sulfates

(> 2,000 ppm), the following service life (SL) equation provides the basis for RCP design in AltPipe:

$$SL = 10^3 \times 1.107^{Cc} \times Cc^{0.717} \times Dc^{1.22} \times (K+1)^{-0.37}$$
$$\times W^{-0.631} - 4.22 \times 10^{10} \times pH^{-14.1} - 2.94 \times 10^{-3}$$
$$\times S + 4.41$$

Where: S = Environmental sulfate content in ppm.

Cc = Sacks of cement (94 lbs each) per cubic yard of concrete.

Dc = Concrete cover in inches.

K = Environmental chloride concentration in ppm.

W = Water by volume as percentage of total mix.

pH = The measure of relative acidity or alkalinity of the soil or water. See Index 855.3.

Where the measured concentration of chlorides exceeds 2000 ppm for RCP that is placed in brackish or marine environments and where the high tide line is below the crown of the invert, the AltPipe input for chloride concentration will default to 25,000 ppm.

Contact the District Materials unit or the Corrosion Technology Branch in DES for design recommendations when in extremely corrosive conditions. Non-Reinforced concrete pipe is not affected by chlorides or stray currents and may be used in lieu of RCP with additional concrete cover and/or protective coatings for sizes 36" in diameter and smaller. See Index 852.1(4) and Table 855.4A. Where conditions occur that RCP designs as produced by AltPipe will not work, the Office of State Highway Drainage Design within the Division of Design should be contacted.

855.5 Material Susceptibility to Fire

Fire can occur almost anywhere on the highway system. Common causes include forest, brush or grass fires that either enter the right-of-way or begin within it. Less common causes include spills of flammable liquids that ignite or vandalism. Storm drains, which are completely buried would typically be impacted by spills or vandalism. Because these are such low probability events, prohibitions on material placement for storm drains are not typically warranted.

Cross culverts and exposed overside drains are the placement types most subject to burning or melting and designers should consider either limiting the alternative pipe listing to non-flammable pipe materials or providing a non-flammable end treatment to provide some level of protection.

Plastic pipe and pipes with coatings (typically of bituminous or plastic materials) are the most susceptible to damage from fire. Of the plastic pipe types which are allowed, PVC will self extinguish if the source of the fire is eliminated (i.e., if the grass or brush is consumed or removed) while HDPE and PP can continue to burn as long as an adequate oxygen supply is present. Based on testing performed by Florida DOT, this rate of burning is fairly slow, and often self extinguished if the airflow was inhibited (i.e., pipe not aligned with prevailing wind or ends sheltered from air flow).

Due to the potential for fire damage, plastic pipe is not recommended for overside drain locations where there is high fire potential (large amounts of brush or grass or areas with a history of fire) and where the overside drain is placed or anchored on top of the slope.

Where similar high fire potential conditions exist for cross culverts, the designer may consider limiting the allowable pipe materials indicated on the alternative pipe listing to non-flammable material types, use concrete endwalls that eliminate exposure of the pipe ends, or require that the end of flammable pipe types be replaced with a length of non-flammable pipe material.

Topic 856 – Height of Fill

An essential aspect of pipe selection is the height of fill/cover over the pipe. This cover dissipates live loads from traffic, both during construction and after the facility is open to the public.

856.1 Construction Loads

See Standard Plan D88 for table of minimum cover for construction loads.

856.2 Concrete Pipe, Box and Arch Culverts

(1) *Reinforced Concrete Pipe.* See Standard Plan A62D and A62DA for the maximum height of overfill for reinforced concrete pipe, up to and including 120-inch diameter (or reinforced oval pipe and reinforced concrete pipe arch with equivalent cross-sectional area), using the backfill method or type shown. For oval shaped reinforced concrete pipe fill heights, see Standard Plan A62D and Indirect Design D-Load (Marsten/Spangler Method). Allowable cover for oval shaped reinforced concrete pipe is determined by using Method 2 (Note 8). See Standard Plan D79 and D79A for pre-cast reinforced concrete pipe Direct Design Method (pertains to circular pipe only).

The designer should be aware of the premises on which the tables on Standard Plan A62D, A62DA, D79 and D79A are computed as well as their limitations. The cover presupposes:

- That the bedding and backfill satisfy the terms of the Standard Specifications, the conditions of cover and pipe size required by the plans, and take into account the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert equal in magnitude to that of the adjoining material outside the trench.
- Subexcavation and backfill as required by the Standard Specifications where unyielding foundation material is encountered.

If the height of overfill exceeds the tabular values on Standard Plan A62D and A62DA a special design is required; see Index 829.2.

(2) *Concrete Box and Arch Culverts.* Single and multiple span reinforced concrete box culverts are completely detailed in the Standard Plans. For cast-in-place construction, strength classifications are shown for 10 feet and 20 feet overfills. See Standard Plan numbers D80, D81 and D82. Pre-cast reinforced concrete box culverts require a minimum of 1 foot overfill and limit fill height to 12 feet maximum. See Standard Plans D83A, D83B and A62G. For fill height design criteria for CIP Bottomless 3-sided rigid frame culverts see DES Section 17 XS-Sheets. Cast-in-place reinforced concrete arch culverts are no longer economically feasible structures and last appeared in the 1997 Standard Plans. Questions regarding fill

height for concrete arch culverts or extensions should be directed to the Underground Structures Branch of DES - Structures Design.

856.3 Metal Pipe and Structural Plate Pipe

Basic Premise - To properly use the fill height design tables, the designer should be aware of the premises on which the tables are based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications and Standard Plan A62F, the conditions of cover, and pipe size required by the plans and the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.

Limitations - In using the tables, the following restrictions must be kept in mind:

- The values given for each size of pipe constitute the maximum height of overfill or cover over the pipe for the thickness of metal and kind of corrugation.
- The thickness shown is the structural minimum. Where abrasive conditions are anticipated, additional metal thickness or invert treatments as stated under Index 852.4(5) and Index 852.6(2)(c) should be provided when required to fulfill the design service life requirements of Topic 855.
- Where needed, adequate provisions for corrosion resistance must be made to achieve the required design service life called for in the references mentioned herein.
- Table 856.3D shows the limit of heights of cover for corrugated steel pipe arches based on the supporting soil sustaining a factored bearing pressure varying between 3.38 tons per square feet to 3.55 tons per square feet. Table 856.3J shows similar values for corrugated aluminum pipe arches.
- The values given for each size of structural plate pipe or arch constitute the maximum height of overfill or cover over the pipe or arch for the thickness of metal and kind of corrugation.
- Tables 856.3N & P show the limit of heights of cover for structural plate arches based on the supporting soil sustaining a factored bearing pressure of 6 tons per square foot at the corners.

Special Designs.

- If the height of overfill exceeds the tabular values, or if the foundation investigation reveals that the supporting soil will not develop the bearing pressure on which the overfill heights for pipe arches are based, a special design prepared by DES - Structures Design is required. See index 829.2.
- Non-standard pipe diameters and arch sizes are available. Loading capacity of special designs needs to be verified with the Underground Structures Branch of DES - Structures Design.
- Aluminum pipe fill height tables are based on use of H-32 temper aluminum. If use of aluminum is necessary and greater structural capacity is required, H-34 temper can be specified. Contact Underground Structures branch of DES-Structures Design for calculation of allowable fill height.

(1) Corrugated Steel Pipe and Pipe Arches, Steel Spiral Rib Pipe, Structural Steel Plate Pipe and Structural Steel Plate Pipe Arches. The allowable overfill heights for corrugated steel

pipe and pipe arches for the various diameters or arch sizes and metal thickness are shown on Tables 856.3A, B, C & D. For steel spiral rib pipe, overfill heights are shown on Tables 856.3E, F, G & H. Table 856.3G gives the allowable overfill height for composite steel spiral rib pipe.

For structural steel plate pipe and structural steel plate pipe arches, overfill heights are shown on Tables 856.3M & N. For maximum height of fill over structural steel plate vehicular undercrossings, see Standard Plan B14-1.

(2) *Corrugated Aluminum Pipe and Pipe Arches, Aluminum Spiral Rib Pipe and Structural Aluminum Plate Pipe and Structural Aluminum Plate Pipe Arches.* The allowable overfill heights for corrugated aluminum pipe and pipe arches for various diameters and metal thickness are shown on Tables 856.3H, I & J. For aluminum spiral rib pipe, overfill heights are shown on Tables 856.3K & L.

For structural aluminum plate pipe and structural aluminum plate pipe arches, overfill heights are shown on Tables 856.3O, & P.

856.4 Plastic Pipe

The allowable overfill heights for plastic pipe for various diameters are shown in Tables 856.4 and 856.5. To properly use the plastic pipe height of fill table, the designer should be aware of the basic premises on which the table is based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications and Standard Plan A62F, the conditions of cover, and pipe size required by the plans and the essentials of Index 829.2.
- That corrugated high density polyethylene (HDPE) and dual wall polypropylene (PP) pipe greater than 48" in size shall be backfilled with cementitious (slurry cement, CLSM or concrete) backfill.
- That where cementitious or flowable backfill is used for structural backfill, the backfill shall be placed to a level not less than 12 inches above the crown of the pipe.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.
- That the average water table elevation is at or below the pipe springline.
- Corrugated HDPE pipe, Type C is recommended for placement only outside the roadbed where vehicular loading is unlikely (e.g., overside drains, medians) unless cementitious backfill is specified.

856.5 Minimum Height of Cover

Table 856.5 gives the minimum thickness of cover required for design purposes over pipes and pipe arches. For construction purposes, a minimum cover of 6 inches greater than the roadway structural section is desirable for all types of pipe.

Table 856.3A

Corrugated Steel Pipe Helical Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)					
	Metal Thickness (in)					
	0.052 (18 ga.)	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)	0.168 (8 ga.)
	2⅔" x ½" Corrugations					
12-15	118	148	177	--	--	--
18	99	124	148	207	--	--
21	85	106	132	177	--	--
24	74	93	116	155	200	245
30	59	74	93	130	160	195
36	49	62	77	108	139	163
42	42	53	66	93	119	139
48	--	46	58	81	104	128
54	--	--	51	72	93	113
60	--	--	--	65	83	102
66	--	--	--	--	76	93
72	--	--	--	--	70	85
78	--	--	--	--	--	75
84	--	--	--	--	--	65
	3" x 1" Corrugations					
48	--	53	67	93	120	147
54	--	47	59	83	107	131
60	--	42	53	75	96	118
66	--	39	48	68	87	107
72	--	35	44	62	80	98
78	--	33	41	57	74	91
84	--	30	38	53	69	84
90	--	28	35	50	64	78
96	--	--	33	47	60	74
102	--	--	31	44	56	69
108	--	--	--	41	53	65
114	--	--	--	39	50	62
120	--	--	--	37	48	59

Table 856.4

Thermoplastic Pipe Fill Height Tables

High Density Polyethylene (HDPE) Corrugated Pipe – Type S

Size (in)	Maximum Height of Cover (ft)
12	15
15	15
18	15
24	15
30	15
36	15
42	15
48	15
54	15
60	15

High Density Polyethylene (HDPE) Corrugated Pipe – Type C

Size(in)	Maximum Height of Cover (ft)
12	5
15	5
18	5
24	5

Polyvinyl Chloride (PVC) Corrugated Pipe with Smooth Interior

Size (in)	Maximum Height of Cover (ft)
12	35
15	35
18	35
21	35
24	35
30	35
36	35

Table 856.4

Thermoplastic Pipe Fill Height Tables (Cont.)

Dual Wall Polypropylene (PP), Corrugated Pipe with Smooth Interior

Size (in)	Maximum Height of Cover (ft)
12	25
15	25
18	25
24	25
30	25
36	20
42	20
48	20
54	20
60	20

Where cover heights above culverts are less than the values shown in Table 856.5, stress reducing slab details available from the Headquarters Design drainage detail library using the following web address may be used: https://design.onramp.dot.ca.gov/drainage-detail-library.

Where cover heights are less than the values shown in the stress reducing slab details, contact Office of State Highway Drainage Design or the Underground Structures Branch of DES - Structures Design.

Topic 857 – Alternate Materials

857.1 Basic Policy

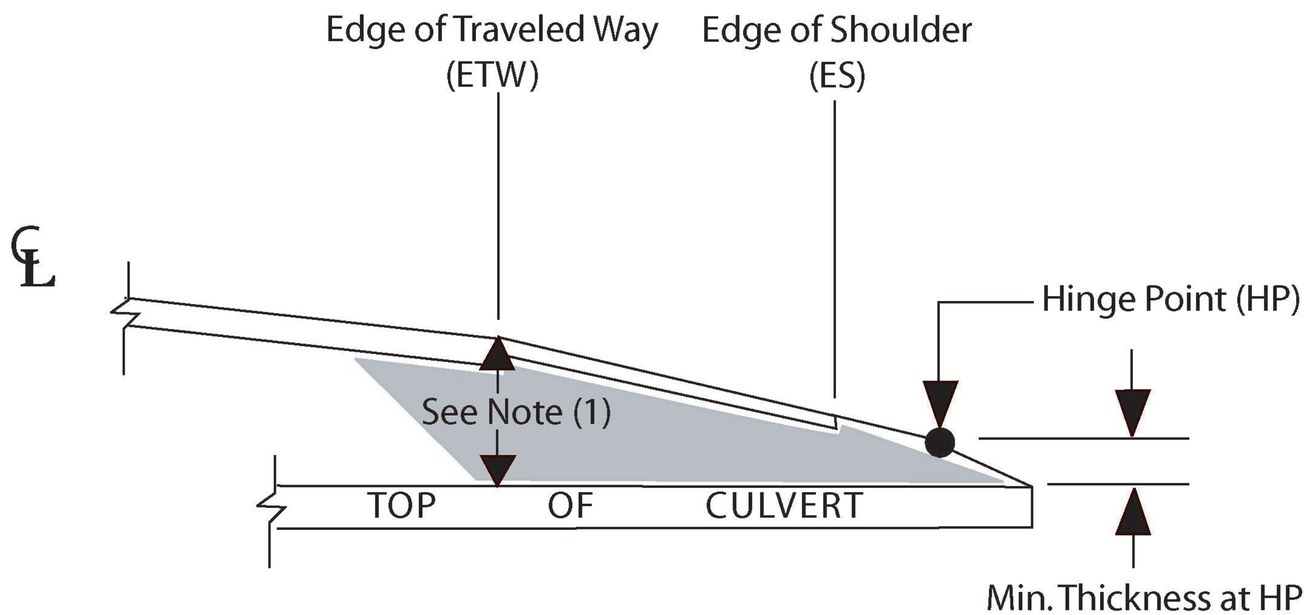
When two or more materials meet the design service life, and structural and hydraulic requirements, the plans and specifications must provide for alternative pipes, pipe arches, overside drains, and underdrains to allow for optional selection by the contractor. See Index 114.3 (2).

(1) Allowable Alternatives. A table of allowable alternative materials for culverts, drainage systems, overside drains, and subsurface drains is included as Table 857.2. This table also identifies the various joint types described in Index 854.1(1) that should be used for the different types of installations.

(2) Design Service Life. Each pipe type selected as an alternative must have the appropriate protection as outlined in Topic 852 to assure that it will meet the design service life requirements specified in Topic 855. The maximum height of cover must be in accordance with the tables included in Topic 856.

Table 856.5

Minimum Thickness of Cover for Culverts



TYPICAL CROSS SECTION

MINIMUM THICKNESS OF COVER AT ETW							
Corrugated Metal Pipes and Pipe Arches	Steel Spiral Rib Pipe	Aluminum Spiral Rib Pipe, S ≤ 48"	Aluminum Spiral Rib Pipe, S > 48"	Structural Plate Pipe	Reinforced Concrete Pipe (RCP) Under Rigid Pavement	RCP Under Flexible Pavement or Unpaved	Plastic Pipes
S/8 or 24" Min.	S/4 or 24" Min.	S/2 or 24" Min.	S/2.75 or 24" Min.	S/8 or 24" Min.	12" Min.	(Max Outside Dimension) /8 or 24" Min.	S/2 or 24" Min.

Notes:
(1) Minimum thickness of cover is measured at ultimate or failure edge of traveled way.
(2) Table is for HL-93 live load conditions only.

"S" in the table is the maximum inside diameter or span of a section.

Table 857.2

Allowable Alternative Materials

Type of Installation	Service Life (yrs)[1]	Allowable Alternatives	Joint Type Standard	Positive	Downdrain
Culverts & Drainage Systems	50	ASSRP, ASRP, CAP, CASP, CSSRP, CIPCP, CSP, NRCP, SAPP, SSPP, SSRP, RCP, RCB, PPC	X	X	--
Overside Drains	50	CAP, CASP, CSP, PPC	--	--	X
Underdrains	50	PAP, PSP, PPET, PPVCP	X	--	--
Arches (Culverts & Drainage Systems)	50	ACSPA, CAPA, CSPA, RCA, SAPPA, SSPPA, SSPA	X	X	--

LEGEND

ACSPA	- Aluminized Corrugated Steel Pipe Arch
ASSRP	- Aluminized Steel Spiral Rib Pipe
ASRP	- Aluminum Spiral Rib Pipe
CAP	- Corrugated Aluminum Pipe
CAPA	- Corrugated Aluminum Pipe Arch
CSSRP	- Composite Steel Spiral Rib Pipe
CASP	- Corrugated Aluminized Steel Pipe, Type 2
CIPCP	- Cast-in-Place Concrete Pipe
CSP	- Corrugated Steel Pipe
CSPA	- Corrugated Steel Pipe Arch
NRCP	- Non-Reinforced Concrete Pipe
PAP	- Perforated Aluminum Pipe
PPC	- Plastic Pipe Culvert
PPET	- Perforated Polyethylene Tubing
PPVCP	- Perforated Polyvinyl Chloride Pipe
PSP	- Perforated Steel Pipe
RCA	- Reinforced Concrete Arch
RCB	- Reinforced Concrete Box
RCP	- Reinforced Concrete Pipe
SAPP	- Structural Aluminum Plate Pipe
SAPPA	- Structural Aluminum Plate Pipe Arch
SSPA	- Structural Steel Plate Arch
SSPP	- Structural Steel Plate Pipe
SSPPA	- Structural Steel Plate Pipe Arch
SSRP	- Steel Spiral Rib Pipe
X	- Permissible Joint Type for the Type of installation Indicated

NOTE:
The design service life indicated for the various types of installations listed in the table may be reduced to 25 years in certain situations. Refer to Index 855.1 for a discussion of service life requirements.

(3) Selection of a Specific Material Type. In the cases listed below, the selection of a specific culvert material must be supported by a complete analysis based on the foregoing factors. All pertinent documentation should be placed on file in the District.

- Where satisfactory performance for a life expectancy of 25 or 50 years, as defined under design service life, cannot be obtained with certain materials by reason of highly corrosive conditions, severe abrasive conditions, or critical structural and construction requirements.
- For individual drainage systems such as roadway drainage systems or culverts which operate under hydrostatic pressure or culverts governed by hydraulic considerations and which would require separate design for each culvert type.
- When alterations or extensions of existing systems are required, the culvert type may be selected to match the type used in the existing system.

857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe

These instructions are general guidelines for alternative pipe culvert selection using the AltPipe computer program that is located on the Headquarters Division of Design alternative pipe culvert selection website at the following web address: https://dot.ca.gov/programs/design/hydraulics-stormwater/bsa-alternative-pipe-culvert-selection-altpipe.

AltPipe is a web-based tool that may be used to assist materials engineers and designers in the appropriate selection of pipe materials for culvert and storm drain applications. The computations performed by AltPipe are based on the procedures and California Test Methods described in this Chapter. AltPipe is not a substitute for the appropriate use of engineering judgment as conditions and experience would warrant. AltPipe establishes uniform procedures to assist the designer in carrying out the majority of the alternative pipe culvert selection functions of the Department, and is neither intended as, nor does it establish, a legal standard for these functions. Implementation of the results and output of this program is solely at the discretion of the user. The user is encouraged to first read the two informational links on the website titled ‘Get More Information’ and ‘How to use Altpipe’ prior to using the program.

Each alternative material selected for a drainage facility must provide the required design service life based on physical and structural factors, be of adequate size to satisfy the hydraulic design, and require the minimum of maintenance and construction cost for each site condition.

Step 1. Obtain the results of soil and water pH, resistivity, sulfate and chloride tests, proposed design life of culverts and make determination if any of the outfalls are in salty or brackish water. The Materials Report should include proposed design life and recommendations for pipe material alternatives. See Indexes 114.2 (3) and 114.3 (2).

Step 2. Obtain hydraulic studies and location data for pipe minimum sizes, and expected Q2-5 flow velocities. For pipes operating under outlet control, a critical element of pipe selection is the Manning’s internal roughness value used in the hydraulic design. It is important to independently verify the roughness used in the design is applicable for the selected alternate materials from AltPipe. Rougher pipes may require larger sizes to provide adequate hydraulic capacities and need steeper slopes to produce desired cleaning velocities, usually however, pipe slope is maintained, and the only variable provided on the plans is pipe size.

Step 3. Determine the abrasion level from Table 852.2A from the maximum size of material that can be moved through a pipe, the expected Q2-5 flow velocities, and Table 855.2B. Field observations of channel bed material both upstream and downstream are recommended.

Step 4. Determine the maximum fill height.

Step 5. Using the AltPipe computer program that is located on the Headquarters Division of Design alternative pipe culvert selection website enter:

- Pipe diameter
- Maximum fill height
- Design service life
- pH
- Minimum resistivity
- Sulfate concentration
- Chloride concentration (for values greater than 2000, check boxes if end of culvert is exposed to brackish conditions and high tide line is below the crown of the culvert)
- Abrasion level
- 2-5-year Storm Flow Velocity (ft/sec)

Repeat step 5 as necessary and save each pipe in worksheet as needed and go to the final summary upon completion.

Step 6. The following alternatives are not included in AltPipe and will not be provided in the output Alternative pipe list: all non-circular shapes (arches, boxes, etc.), non reinforced concrete pipe (NRCP) and non-standard new products. Check Materials and Hydraulics reports and verify if any of these alternatives were recommended and supplement the AltPipe final summary accordingly. For reinforced concrete pipe (RCP), box (RCB) and arch (RCA) culverts, maintenance-free service life, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of exposed reinforcement at any point on the culvert. Changes in the design may be required in relatively severe acidic, chloride or sulfate environments. The levels of these constituents (either in the soil or water) will need to be identified in the project Materials or Geotechnical Design Report. The adopted procedure consists of a formula that the constituent concentrations are entered into in order to determine a pipe service life. The means for offsetting the affects of the corrosive elements is to increase the cover over the reinforcing steel, increase the cement content, or reduce the water/cement ratio.

Step 7. Table 855.2C constitutes a guide for abrasive resistant coatings in low to moderate abrasive conditions for metal pipe (i.e., Levels 1 through 5 in Table 855.2A) and is included in AltPipe. Table 855.2F constitutes a guide for minimum material thickness of abrasive resistant invert protection to achieve 50 years of maintenance-free service life in moderate to highly abrasive conditions (i.e., Levels 4 through 6 in Table 855.2A) and was not programmed into AltPipe. If pipe material thickness does not meet service life due to abrasive conditions, consideration for invert protection should be made using Table 855.2F as a guide.

857.3 Alternative Pipe Culvert (APC) and Pipe Arch Culvert List

Because of the difference in roughness coefficients between various materials, it may be necessary to specify a different size for each allowable material at any one location. In this event, it is recommended that the material with the smallest dimension be listed as the alternative size. Refer to Plans Preparation Manual for standard format to be used.

There may be situations where there is a different set of alternatives for the same nominal size of alternative drainage facilities. In this case the different sets of the same nominal size should be further identified by different types, for example, 18-inch alternative pipe culvert (Type A), 18-inch alternative pipe culvert (Type B), etc. No attempt to correlate type designation between projects is necessary. The first alternative combination for each culvert size on each project should be designated as Type A, second as Type B, etc.

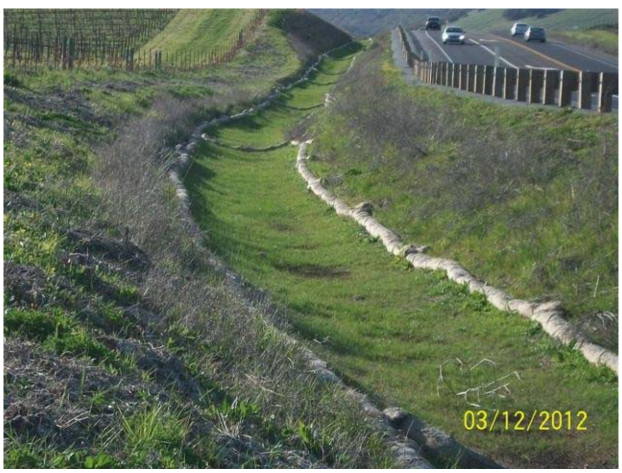
Since the available nominal sizes for pipe arches vary slightly between pipe arch materials, it is recommended that the listed alternative pipe arch sizes conform to those sizes shown for corrugated steel pipe arches shown on Table 856.3D. The designer should verify the availability of reinforced concrete pipe arches. If reinforced concrete pipe arches are not available, oval shaped reinforced concrete pipe of a size necessary to meet the hydraulic requirements may be used as an alternative.

etc. Typical flexible lining materials include grass or small-rock slope protection, while typical rigid lining materials include hot mixed asphalt or Portland cement concrete. Flexible linings are generally less expensive, may have a more natural appearance, permit infiltration and exfiltration and are typically more environmentally acceptable. Vegetative channel lining is also recognized as a best management practice for storm water quality design in highway drainage systems. A vegetated channel helps to deposit highway runoff contaminants (particularly suspended sediments) before they leave the highway right of way and enter streams. See Index 861.11 'Water Quality Channels' and Figure 865.1.

On steep slopes, most vegetated flexible linings are limited in the erosive forces they can sustain without damage to the channel and lining unless the vegetative lining is combined with another more erosion-resistant long-term lining below, such as a cellular soil confinement system. See Figure 865.1 and Index 865.3(1). The District Landscape Architect should be contacted to provide viable vegetation alternatives within the District, however all design responsibilities belong to the Project Engineer.

Figure 865.1

Steep-Sloped Channel with Composite Vegetative Lining



Vegetative flexible lining placed on top of cellular soil confinement system on a steep-sloped channel.

865.2 Rigid

A rigid lining can typically provide higher capacity and greater erosion resistance and in some cases may be the only feasible alternative.

Rigid linings are useful in flow zones where high shear stress or non-uniform flow conditions exist, such as at transitions in channel shape or at an energy dissipation structure.

The most commonly used types of rigid lining are hot mixed asphalt and Portland cement concrete. Hot mixed asphalt is used mainly for small ditches, gutters and overside drains (see Standard Plan D87D) because it cannot withstand hydrostatic pressure from the outside.

Table 865.1 provides a guide for Portland cement concrete and air blown mortar roadside channel linings. See photo below Table 865.1 for example.

For the design of concrete lined flood control channels discussed in Index 861.2 (1), see U.S. Army Corps of Engineers publication; “Structural Design of Concrete Lined Flood Control Channels”, EM 1110-2-2007:

http://planning.usace.army.mil/toolbox/library/EMs/em1110.2.2007.pdf

Table 865.1

Concrete[2] Channel Linings

Abrasion Level[1]	Thickness of Lining (in)		Minimum Reinforcement
	Sides	Bottom	
1 - 3	5	5	6 x 6-W2.9 x W2.9 welded wire reinforcement

NOTES:

[1]See Table 855.2A.

[2]Portland Cement Concrete or Air Blown Mortar

Figure 865.2

Concrete Lined Channel

For large flows, consideration should be given to using a minimum bottom width of 12 feet for construction and maintenance purposes, but depths of flow less than one foot are not recommended. Despite the non-erodible nature of rigid linings, they are susceptible to failure from foundation instability and abrasion. The major cause of failure is undermining that can occur in a number of ways.

CHAPTER 870 – BANK PROTECTION – EROSION CONTROL

Topic 871 – General

Index 871.1 – Introduction

Highways, bikeways, pedestrian facilities and appurtenant installations are often attracted to parallel locations along man-made channels, streams, and rivers. These locations may be affected from the action of flowing water, and may require protective measures.

Bank protection can be a major element in the design, construction, and maintenance of highways. This section deals with procedures, methods, devices, and materials commonly used to mitigate the damaging effects of flowing water on transportation facilities and adjacent properties. Potential sites for such measures should be reviewed in conjunction with other features of the project such as long and short term protection of downstream water quality, aesthetic compatibility with surrounding environment, and ability of the newly created ecological system to survive with minimal maintenance. See Index 110.2 for further information on water quality and environmental concerns related to erosion control. See Chapter 880 for shore protection along coastal zones and lake shores that are subjected to wave attack.

Refer to Index 806.2 for definitions of drainage terms.

871.2 Design Philosophy

In each district there should be a designer or advisor, usually the District Hydraulic Engineer, knowledgeable in the application of bank protection principles and the performance of existing works. Information is also available from headquarters specialists in the Division of Design and Structures Design in the Division of Engineering Services (DES). The most effective designs result from the cooperative involvement with Design, Environmental, Landscape Architecture, Structures, Construction, and Maintenance (for further discussion on functional responsibilities see Topic 802). For channel and habitat characterization and assessment relative to design and obtaining project specific permits, the designer may also require input from fluvial geomorphologists (or engineers with geomorphology training), geologists and biologists. The District Hydraulic Engineer will typically be able to assist with flood analysis, water surface elevations/profiles, shear stress computations, scour analysis, and hydraulic analysis for placement of in-stream structures. A geomorphologist can provide input regarding characterization of channel form and dominant geomorphic processes and hydraulic geometry relationships such as an analysis of lateral and longitudinal channel adjustment. The geomorphologist can also make an identification of the processes responsible for forming and maintaining key habitats and assist in making an assessment of the long-term project effects. See Index 872.3 for project geomorphologist details.

There are a number of ways to deal with the problem of bank erosion as follows:

- Although not always feasible or economical, the simplest way and generally the surest of success and permanence, is to locate the facility away from the erosive forces. Locating the facility to higher ground or solid support should be considered, even when it requires excavation of solid rock, since excavated rock may serve as a valuable material for bank protection.
- The most commonly used method of bank protection is with a more resistant material like rock slope protection (RSP). Other protection methods (e.g., training systems) are discussed in Index 873.4 and summarized in Table 872.1.
- A third method is to reduce the force of the attacking water. This is often done by various plantings such as willows. Plantings once established not only reduce stream velocity near the bank during heavy flows, but their roots add structure to the bank material.
- Another method is to re-direct flows away from the embankment. In the case of stream attack, a new channel can be created or the stream can be diverted away from the embankment by the use of baffles, deflectors, or spurs.

Combinations of the above four methods may be used. Even protective works destroyed in floods have proven to be effective and cost efficient in minimizing damage to transportation facilities.

Design of protective features should be governed by the importance of the facility and appropriate design principles. Some of the factors which should be considered are:

- *Roughness*. Revetments generally are less resistant to flow than the natural channel bank. Channel roughness can be significantly reduced if a rocky vegetated bank is denuded of trees and rock outcrops. When a rough natural bank is replaced by a smooth revetment, the current is accelerated, increasing its power to erode, especially along the toe and downstream end of the revetment. Except in narrowed channels, protective elements should approximate natural roughness and simulate the effect of trees and boulders along natural banks and in overflow channels.
- *Undercutting*. Particular attention must be paid to protecting the toe of revetments against undercutting caused by the accelerated current along smoothed banks, since this is the most common cause of bank failure.
- *Standardization*. Standardization should be a guide but not a restriction in designing the elements and connections of protective structures.
- *Expendability*. The primary objective of the design is the security of the transportation facility, not security of the protective structure. Less costly replaceable protection may be more economical than expensive permanent structures.
- *Dependability*. An expensive structure is warranted primarily where transportation facilities carry high traffic volumes, where no reasonable detour is available, or where facility replacement is very expensive.
- *Longevity*. Short-lived structures or materials may be economical for temporary situations. Expensive revetments should not be placed on banks likely to be buried in widened embankments, nor on banks attacked by transient meander of mature streams.
- *Rock Materials*. Optimum use should be made of local materials, considering the cost of special handling. Specific gravity of stone is a major factor in bank protection and the specified minimum should not be lowered without increasing the mass of stones. See Index 873.3(3)(a)(2)(b) for equation to estimate rock size.

- *Selection*. Selection of class and type of protection should be guided by the intended function of the installation.
- *Limits*. Horizontal and vertical limits of protection should be carefully designed. The bottom limit should be secure against toe scour. The top limit should not arbitrarily be at high-water mark, but above it if overtopping would cause excessive damage and below it if floods move slowly along the upper bank. The end limits should reach and conform to durable natural features or be secure with respect to design parameters.

Table 872.1

Guide to Selection of Protection

Location	Armor											Training										
	Flexible					Rigid						Guide Banks				Bendway Weirs and Spurs				Check Dams		
				Mattresses																		
	Vegetation	Rip Rap	Vegetated RSP	Gabions	Conc. F	Rock	Grouted Rock	Conc. Rock	Conc. Lined	Cribs	Bulk Heads	Earth	Rock	Piling	Other	Rock	Grouted Rock	Piling	Other	Drop Structure	Piling	Rock
Cross Channel																						
Young Valley		X					X	X		X	X											
Mature Valley		X					X	X		X	X	X		X	X	X	X			X		X
Parallel Encroachment																						
Young Valley		X	X				X	X		X	X											
Mature Valley	X	X	X				X	X		X	X	X		X	X	X	X	X	X	X		X
Desert-wash																						
Top debris cone		X					X	X			X	X										
Center debris cone		X					X	X												X		X
Bottom debris cone		X					X	X												X		X
Overflow and Floodplain	X	X	X				X					X		X	X							
Artificial Channel or Roadside Ditch (Ch. 860)	X	X	X	X	X	X	X	X	X													
Culvert																						
Inlet		X					X				X											
Outlet		X					X				X											
Bridge																						
Abutment		X					X		X													
Upstream		X					X					X	X	X	X							
Downstream		X					X					X	X	X	X					X	X	X

871.3 Selected References

Hydraulic and drainage related publications are listed by source under Topic 807. References specifically related to slope protection measures are listed here for convenience.

(a) FHWA Hydraulic Engineering Circulars (HEC) – The following seven circulars were developed to assist the designer in using various types of slope protection and channel linings:

- HEC 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (2006)
- HEC 15, Design of Roadside Channels with Flexible Linings (2005).
- HEC 18, Evaluating Scour at Bridges (2012)
- HEC 20, Stream Stability at Highway Structures (2012)
- HEC 23, Bridge Scour and Stream Instability Countermeasures (2009)
- HEC 25, Highways in the Coastal Environment (2008 with 2014 supplement)
- HEC 26, Culvert Design for Aquatic Organism Passage (2010)

(b) FHWA Hydraulic Design Series (HDS) No. 6, River Engineering for Highway Encroachments (2001) – A comprehensive treatise of natural and man-made impacts and responses on the river environment, sediment transport, bed and bank stabilization, and countermeasures.

(c) AASHTO Highway Drainage Guidelines – General guidelines for good erosion control practices are covered in Volume III - Erosion and Sediment Control in Highway Construction

(d) AASHTO Drainage Manual (2014) – Refer to Chapters; 11 – Energy Dissipators; 16 – Erosion and Sediment Control; 17 – Bank Protection. The manual provides guidance on engineering practice in conformance with FHWA's HEC and HDS publications and other nationally recognized engineering policy and procedural documents.

(e) U.S. Army Corps of Engineers EM 1110-2-1601 Hydraulic Design of Flood Control Channels Manual.

(f) California Department of Fish and Wildlife – California Salmonid Stream Habitat Restoration Manual.

(g) FHWA Reference Document (2019) - Two-Dimensional Hydraulic Modeling for Highways in the River Environment.

Topic 872 – Planning and Location Studies

872.1 Planning

The development of sustainable, cost effective and environmentally friendly protective works requires careful planning and a good understanding of both the site location and habitat within the stream reach and overall watershed. Planning begins with an office review followed by a site investigation.

Google Earth can be a useful tool for determining site location, changes to stream planform (pattern), bend radius to channel width ratio (to estimate rock size per Index 873.3(3)(a)(2)(b), and location within the overall watershed. USGS StreamStats will facilitate simple watershed delineation and provide basin characteristics such as area, cover and percentage of impervious cover, average elevation, stream slope, mean annual precipitation, and peak flow from regression equations. When more detailed watershed delineation is required, United States Geological Survey (USGS) 7.5-minute quadrangle maps are used to trace the tributary area and sub-basins. The USGS maps are found in graphic image form, such as TIFF and JPEG, and are also found in the form of a Digital Elevation Model (DEM). A DEM contains x-y-z topographic data point usually at 1, 10 or 30-meter grid intervals, where ”x” and “y” represent horizontal position coordinates of a topographic point and “z” is its elevation. These data files and the USGS 7.5-minute quadrangle image files can be imported into software programs, including the Watershed Modeling System (WMS), Surface-water Modeling System (SMS), AutoCAD Civil 3D, and ArcGIS.

Nearby bridges that are located along the same stream reach should be reviewed for site history and changes in stream cross-section. All bridge files of existing bridges are located in the Division of Maintenance, Office of Structures Maintenance.

District biologist staff should be consulted early on during the project planning phase for subject matter expertise regarding fisheries, habitat, and wildlife and to perform an initial stream habitat assessment.

Department biologists can be accessed through the Project Delivery Team's Environmental Coordinator, or DEA's Biological Services Office.

For channel and habitat characterization and preliminary assessment relative to design and acquisition of project specific permits, the initial site investigation team should include the project engineer, the district hydraulic engineer, and a biologist. Depending on the complexity of the project, it may be necessary to include Caltrans staff that are trained to perform a geomorphic assessment and/or a geologist during the site investigation.

The selection of the type of protection can be determined during or following the site investigation. For some sites the choice is obvious; at other sites several alternatives or combinations may be applicable. See the FHWA’s HDS No. 6, River Engineering for Highway Encroachments for a complete and thorough discussion of hydraulic and environmental design considerations associated with hydraulic structures in moveable boundary waterways.

Some specific site conditions that may dictate selection of a type of protection different from those shown in Table 872.1 are:

- Available right of way.
- Available materials.
- Possible damage to other properties through streamflow diversion or increased velocity.
- Environmental concerns.
- Channel capacity or conveyance.
- Conformance to new or existing structures.
- Provisions for side drainage, either surface waters or intersecting streams or rivers.

The first step is to determine the limits of the protection with respect to length, depth and the degree of security required. For more detailed stream reconnaissance considerations, see HEC 20, Index 4.2.1 (Appendix C and D) and the FHWA’s HDS No. 6, River Engineering for Highway Encroachments (Table 8.1).

Considerations at this stage are:

- The severity of stream attack.
- The present alignment of the stream or river and potential meander changes.
- The ratio of cost of highway replacement versus cost of protection.

- Whether the protection should be permanent or temporary.
- Analysis of foundation and materials explorations.
- Access for construction.
- Bank slope (H:V).
- Bed and bank material gradations.
- Stream stability (lateral and vertical). Caltrans Hydromodification Requirements Guidance Storm Water Best Management Practices Rapid Assessment of Stream Crossings Higher Level Stream Stability Analysis presents 13 channel characteristics that are indicators of present stream stability. See Index 4.1.
- Local stream profile.
- Vegetation type and location.
- Physical habitat (temperature, shade, pools, riffles, sediment supply).
- Toe scour/bank failure mode (see Table 872.2).
- Thalweg location.
- Hardpoint location(s).
- Total length of protection needed.

The second step is the selection and layout of protective elements in relation to the highway facility.

872.2 Class and Type of Protection

Protective devices are classified according to their function. They are further categorized as to the type of material from which they are constructed or shape of the device. For additional information on specific material types and shapes see Topic 873, Design Concepts.

There are two basic classes of protection, armor treatment and training works. Table 872.1 relates different location environments to these classes of protection.

872.3 Geomorphology and Site Consideration

The determination of the lengths, heights, alignment, and positioning of the protection are affected to a large extent by the facility location environment.

An evaluation is required for any proposed highway construction or improvement that encroaches on a floodplain. See Topic 804, Floodplain Encroachments for detailed procedures and guidelines.

(1) Geomorphology. An understanding of stream morphology is important for identifying both stream instability and associated habitat problems at highway-stream locations. A study of the plan and profile of a stream is very useful in understanding stream morphology. Plan view appearances of streams are varied and result from many interacting variables. Small changes in a variable can change the plan view and profile of a stream, adversely affecting a highway crossing or encroachment. This is particularly true for alluvial streams. Conversely, a highway crossing or encroachment can inadvertently change multiple variables such as Manning's "n-value", channel width, and average velocity, which may adversely affect the stream.

There is not a legal definition of geomorphologist or geomorphology under California regulations. The Project Geomorphologist can be one person or a team of a few professionals (Civil engineers, Geologists, and Engineering Geologists, and/or Geotechnical Engineers) involved in the project development, with experience in fluvial geomorphology as described above in this chapter and referenced materials. Geomorphologist design work falls under the umbrella of civil engineer in the California Business and Professional Code, Section 6731, drainage, flood control, bridges, and natural inland waterways. Geomorphology on Caltrans drainage projects should be under the responsible charge of a Civil Engineer with background in hydrology, hydraulics, sediment transport, geology and how these engineering forces interact with the natural waterways including the vegetation to create or change landforms.

Chapter 2 in HEC 20 presents an overview of general landform and channel evolutionary processes to illustrate the dynamics of alluvial channel systems. It discusses lateral stability, factors effecting bed elevation changes, and the sediment continuity principle to provide an introduction to alluvial channel response to natural and human-induced change.

River morphology and river response is discussed in detail in Chapter 5 of FHWA's HDS No. 6, River Engineering for Highway Encroachments.

(2) *Stream Processes.* Prior to the current interest in ecology, water quality, and the environment, few engineers involved with highway crossings and encroachments considered the short-term and long-term changes that were possible or the many problems that humans can cause to streams. It is imperative that anyone working with rivers, either on localized areas or entire systems, have an understanding of the many factors involved, and of the potential for change within the river system. Highway construction can have significant general and local effects on the geomorphology and hydraulics of river systems. Hence, it is necessary to consider induced short-term and long-term effects of erosion and sedimentation on the surrounding landscape and the river. The biological response of the river system should also be considered and evaluated. Certain species of fish can only tolerate large concentrations of suspended sediment for relatively short periods of time. This is particularly true of the eggs and fry. It is useful for the project engineer to understand what is important for regulators. Some of the most common topics include:

- Site geomorphology and steam stability
- Stressors to historic aquatic organism habitat
- Locations of hydraulic constrictions

Only with such knowledge can the project engineer develop the necessary arguments to make the case that erosion control measures must be designed to avoid significant deterioration of the stream environment not only in the immediate vicinity of the highway encroachment or crossing, but in many instances for great distances downstream.

Fluvial geomorphology is the science dealing with the shape of stream channels and includes the study of physical processes within river systems, such as bank erosion, sediment transport, and bed material sorting.

This section is intended to give the engineer background, perspective, and respect of stream processes and their dynamics when designing and constructing bank protection for natural streams and to lay the groundwork for application of the concepts of open-channel flow, fluvial geomorphology, sediment transport, and river mechanics to the design, maintenance, and environmental challenges associated with highway crossings and encroachments. Encroachment is any occupancy of the river and floodplain for highway use. Encroachments usually present no issues during normal stages, but require special protection against floods. Classifying the regions requiring protection, the possible types of protection, the possible flow conditions, the possible channel shapes, and the various geometric conditions aids the engineer in selecting the design criteria for the conditions encountered.

(a) Types of Encroachment. In the vicinity of rivers, highways generally impose a degree of encroachment. In some instances, particularly in mountainous regions or in river gorges and canyons, river crossings can be accomplished with absolutely no encroachment on the river. The bridge and its approaches are located far above and beyond any possible flood stage. More commonly, the economics of crossings require substantial encroachment on the river and its floodplain, the cost of a single span over the entire floodplain tends to be prohibitive. The encroachment can be in the form of earth fill bridge approach embankments on the floodplain or into the main channel itself, reducing the required bridge length; or in the form of piers and abutments or culverts in the main channel of the river. Longitudinal encroachments may exist that are not connected with river crossings. Floodplains often appear to provide an attractive low cost alternative for highway location, even when the extra cost of flood protection is included. As a consequence, highways, including interchanges, often encroach on a floodplain over long distances. In some regions, such as mountainous regions, river valleys (or canyons) provide the only feasible route for highways. This is true in areas where a floodplain does not exist. In many locations the highway encroaches on the main channel itself and the channel is partly filled to allow room for the roadway. See Figure 872.4. In some instances, this encroachment becomes severe, particularly as older highways are upgraded and widened.

(b) Effects of River Development Works. These works may include water diversions to and from the river system, dams, cutoffs (channel straightening), levees, navigation works, and the mining of sand and gravel. It is essential to consider the probable long-term plans of all agencies and groups as they pertain to a river when dealing with the river in any way. For example, dams serve as traps for the sediment normally flowing through the river system. With sediment trapped in the reservoir, essentially clear water is released downstream of the dam site. This clear water has the capacity to transport more sediment than may be immediately available. Consequently the channel begins to supply this deficit with resulting degradation of the bed or banks. The degraded or widened main channel causes steeper gradients on tributary streams in the vicinity of the main channel. The result is degradation in the tributary streams. It is entirely possible, however, that the additional sediments supplied by the tributary streams would ultimately offset the degradation in the main channel. Thus, it must be recognized that downstream of storage structures the channel may either aggrade or degrade (most common) and the tributaries will be affected in either case.

(c) Alluvial Streams. Most streams that highways cross or encroach upon are alluvial; that is, the streams are formed in materials that have been and can be transported by the stream. In alluvial stream systems, it is the rule rather than the exception that banks will erode; sediments will be deposited; and floodplains, islands, and side channels will undergo modification with time. Alluvial channels continually change position and shape as a consequence of hydraulic forces exerted on the bed and banks. These changes may be gradual or rapid and may be the result of natural causes or human activities. At any location in a stream, the cross-sectional shape is dependent upon the volume flow-rate (flow), the composition of sediment transported through a section, and the integrity or gradation of the bed and bank materials. As water flows through the stream channel, it exerts a fluid shear stress on the bed and banks. For a constant and stable cross-sectional shape for a given flow at a specific location, the resisting bed and bank material shear stress must be equal to the fluid stress at every point in the stream cross section perimeter. In this state, a stream is in the threshold condition where each point along the perimeter is at the threshold of movement or incipient motion. This condition also indicates a dynamic equilibrium with scour and deposition of sediment being equal. As flow, velocity, and fluid shear stress increase, the amount of scour and sediment deposition will change, which will also change the stream cross section for a given bed/bank gradation.

Alluvial streams are commonly trapezoidal in cross section through their straight reaches and become asymmetric through their bends. When streams incise in response to possible instability, their depth increases and the stream takes on a more rectangular cross-sectional shape. Also, streams with very large flows may become rectangular as the bed width increases to convey the large flows, especially if bedrock outcroppings are present on the banks preventing them from flattening.

(d) Non-Alluvial Streams. Some streams are not alluvial. The bed and bank material is very coarse, and except at extreme flood events, do not erode. These streams are classified as sediment supply deficient, i.e., the transport capacity of the streamflow is greater than the availability of bed material for transport. The bed and bank material of these streams may consist of cobbles, boulders, or bedrock. In general these streams are stable, but should be carefully analyzed for stability at large flows. A study of the plan and profile of a stream is useful in understanding stream morphology. Plan view appearances of streams are varied and result from many interacting variables. Small changes in a variable can change the plan view and profile of a stream, adversely affecting a highway crossing or encroachment. This is particularly true for alluvial streams. Conversely, a highway crossing or encroachment can inadvertently change a variable, adversely affecting the stream.

(e) Dynamics of Natural Streams. Long-term climatic and tectonic fluctuations have caused major changes of river morphology, but rivers can display a remarkable propensity for change of position and morphology in time periods of a century. For shorter time periods river channels will shift through erosion and deposition at bends and may form chutes, islands or oxbow lakes. Lateral migration, erosion and deposition rates are not linear; i.e., a river may maintain a stable position for several years and then experience rapid movement. At low flow the bed of a sand bed stream can be dunes, but at large flows the bed may become plane or have antidune flow. With dunes, resistance to flow is large and bed material transport is low. Whereas, with plane bed or antidune flow the resistance to flow is small and the bed material transport is large. Much, therefore, depends on flood events, bank stability, permanence of vegetation on banks and the floodplain and watershed land use.

In summary, archaeological, botanical, geological, and geomorphic evidence supports the conclusion that most rivers are subject to constant change as a normal part of their morphologic evolution. Therefore, stable or static channels are the exception in nature.

If an engineer modifies a river channel locally, if not designed properly, this local change may cause unintended modification of channel characteristics both up and down the stream. The response of a river to human-induced changes often occurs in spite of attempts by engineers to keep the anticipated response under control. The points that should be stressed are that a river through time is dynamic and that human-induced change frequently sets in motion a response that can be propagated upstream or downstream for long distances.

In spite of their complexity, all rivers are governed by the same basic forces. The design engineer must understand, and work with these natural forces:

- Geological factors, including soil and seismic conditions.
- Hydrologic factors, including possible changes in flows, runoff, and the hydrologic effects of changes in land use.

- Geometric characteristics of the stream, including the probable geometric alterations that will be activated by the changes a project and future projects will impose on the channel.
- Hydraulic characteristics such as depths, slopes, and velocity of streams and what changes may be expected in these characteristics in space and time.
- Sea level rise may also cause river instability, particularly when the 75-year design life of a bridge is considered.

(f) Basic Stream Pattern. The three basic stream patterns are straight, braided, and meandering as seen in aerial or plan view. Pattern is one way of classifying a stream and generalizing its behavior, another is sediment load. See Figure 872.1.

Commonly, stream patterns are identified by sinuosity, which is defined as channel length divided by valley (floodplain) length. For straight and braided streams, sinuosity varies between 1.0 and 1.5, while meandering streams have sinuosity greater than 1.5. These different patterns and their associated gradients contribute to changes and adjustments in streams, and specifically influence flow resistance that effects sediment transport and formation of cross-sectional shape. Engineers using any stream classifications should be aware that they are artificial constructs, and no strict science laws or principles of classification (such as used in biology) are possible. Although we may assign channel reaches to discrete categories based on arbitrary thresholds of slope, sinuosity, bed material size, sediment load, width-depth ratio, etc., these quantities vary continuously, and channels tend to behave in rather individualistic fashion. Different types of streams occur within a given subregion. Caltrans Hydromodification Requirements Guidance presents the various stream forms within each of the physiographic subregions of California, and is available from the Headquarters Office of Hydraulics and Stormwater Design.

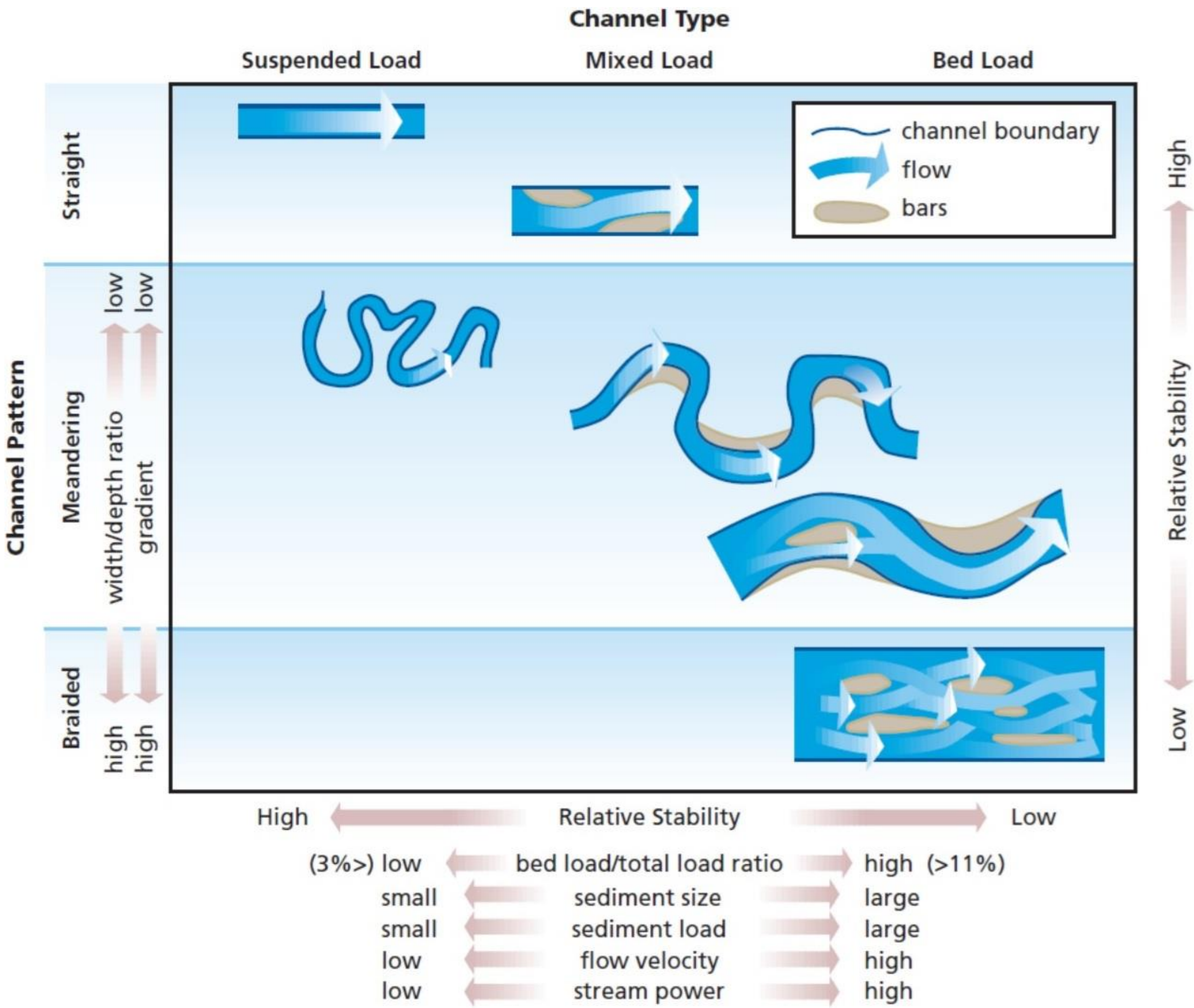
(g) Straight Streams. Straight river channels can be of two types. The first forms on a low-gradient valley slope, has a low width-depth ratio channel, and is relatively stable. The first type of straight channel may contain alternate islands or bars that result in a sinuous thalweg (flow path connecting deepest points in successive cross sections) within the straight channel. It may seem that the first type of straight stream is very stable because of low slope and energy, but alternating sediment deposits can cause lateral instability. In general, it is more natural for a stream to meander than to have a straight stream pattern, therefore it is difficult to find low-gradient straight streams in the field, especially long reaches.

The second type is a steep gradient, high width-depth ratio, high energy river that has many islands or bars, and at low flow is braided. It is relatively active.

In general, the designer should not attempt to develop straight channels fully protected with riprap. In a straight channel the alternate islands or bars and the thalweg are continually changing; thus, the current is not uniformly distributed through the cross-section but is deflected toward one bank and then the other.

Figure 872.1

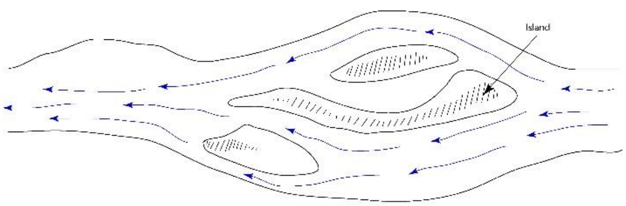
Stream Classification



(h) Braided Streams. Similar to straight streams, streams with braided pattern have low sinuosity, but have the highest gradient of any of the stream patterns. Braided streams have many sub-channels within the main stream channel that interweave and crisscross. The sub-channels are separated by islands or bars which are visible during low flows and normally submerged under high flows. Because braided streams have steep slopes, they possess the higher energy necessary to erode and transport sediment that comprises the bars and islands. Even though braided streams have high energy, these streams will deposit their coarser and larger material that cannot be physically transported by the stream's average velocity and shear stress. In other words, the process of braiding occurs during flood events as a stream adjusts in response to the larger sediment and debris loads that cannot be sustained while trying to find dynamic equilibrium. This deposition of larger material creates the bars and islands. See Figure 872.2. As flow and velocity fluctuate during a flood event, it is common to see movement and re-creation of bars, islands, and sub-channels.

Figure 872.2

Diagram of a Braided River Channel



(i) Meandering Streams. Meandering is the most common stream pattern, having a series of alternating curves or bends, and is associated with flatter valleys. Meandering stream types have the highest sinuosity because of their longer stream length, due to several alternating curves, with respect to valley length, see Figure 872.6. One way that streams seek dynamic equilibrium is to dissipate energy through erosion of their banks, creating meandering patterns. When meanders are created, overall stream length is increased, and energy is released through the work necessary to scour its banks, which brings a stream closer to dynamic equilibrium. Streambank revetments are often constructed through these meanders to prevent excessive erosion that may cause instability of nearby or adjacent transportation facilities.

Once curves have been created in a stream's alignment, velocity increases as the flow of water moves through the outside bank of a bend caused by secondary circulation currents. Given the geometry of a curve, velocity is resolved into three components described in the longitudinal, width-wise, and vertical directions, contrary to straight reaches of stream.

As flow moves through a curve, the circulation currents and their turbulence are influenced by radius of curvature, stream bottom width, flow depth, curve deflection angle, and Reynolds Number. As often occurs, turbulence is magnified by counter-circulating currents from an upstream bend merging with circulating currents of an immediate downstream bend. The increased turbulence usually increases the amount of scour at the outside bend, and the transported material is deposited on the inside bend at the downstream reversing curve creating a point island or bar.

Another characteristic of flow through a curve is that the top of the water surface will superelevate along the outside bank of a curve as it is pulled by centrifugal forces while the bottom water surface at the bed is being pulled toward the inside of a bend. These two actions will cause skewing of the circulating current contributing to increased erosion around a bend.

(j) Sediment Transport. For engineering purposes, the two sources of sediment transported by a stream are: (1) bed material that makes up the stream bed; and (2) fine material that comes from the banks and the watershed (washload). Geologically both materials come from the watershed, but for the engineer, the distinction is important because the bed material is transported at the capacity of the stream and is functionally related to measurable hydraulic variables. The washload is not transported at the capacity of the stream. Instead, the washload depends on availability and is not functionally related to measurable hydraulic variables.

The division size between washload and bed sediment load is sediment size finer than the smallest 10 percent of the bed material. It is important to note that in a fast flowing mountain stream with a bed of cobbles the washload may consist of coarse sand sizes. For these conditions, the transport of sand sizes is supply limited. In contrast, if the bed of a channel is silt, the rate of bed load transport of the silt sizes is less a question of supply than of capacity.

When a river reaches equilibrium, its transport capacities for water and sediment are in balance with the rates supplied. In fact, most rivers are subject to some kind of control or disturbance, natural or human-induced that gives rise to non-equilibrium conditions.

HDS No. 6, Index 4.3.2, states total sediment load can be expressed by three equations:

1. By type of movement

$$L_T = L_b + L_s$$

2. By method of measurement

$$L_T = L_m + L_u$$

3. By source of sediment

$$L_T = L_w + L_{bm}$$

Where:

L_T = Total load;

L_b = Bed load which is defined as the transport of sediment particles that are close to or maintain contact with the bed;

L_s = Suspended load defined as the suspended sediment passing through a stream cross-section above the bed layer;

L_m = Measured sediment;

L_u = Unmeasured sediment that is the sum of bed load and a fraction of suspended load below the lowest sampling elevation;

L_w = Wash load which is the fine particles not found in the bed material ($Ds < D_{10}$), and originates from available bank and upstream supply;

L_{bm} = Capacity limited bed material load.

Streams are unique from other hydraulic conveyance facilities, such as engineered channels and pipes, in that its boundaries are mobile, and they move sediment within their water column or along the bed by skipping and rolling, which is a complicated interrelationship. The suspended sediment load is carried through the flow by turbulence and is typically fine sand, silt, and clay. Bedload is coarser possibly as large as boulders, and moves along the bed by fluid stress action, see Figure 872.3. Sediment supply and its movement are the life of a stream that can become unstable when this process is interrupted if supply becomes limited or if a stream is unable to transport its excess downstream.

Instability can be seen through channel incision, where the stream bed degrades and banks over steepen, excessive meandering, or large alignment shifts as a stream attempts to control energy as it searches for dynamic equilibrium. The ability of a stream to control and manage its sediment is not the only influence on stream stability, but one of the more important factors.

Within a stream bed, immersed sediment particles resting on the stream bed over other particles exert their effective weight in the form of a vertical force, which can be divided into normal and tangential components based on the stream bed slope. Simply stated, in order for sediment particles to become mobile, a force greater than their normal weight must be applied to them. This force that causes mobility is a drag force or fluid stress acting on the particle as water flows over them. The fluid stress can be expressed as an average boundary shear stress acting on a stream bed considering steady, uniform flow:

$$\sigma_0 = \gamma_w D S_f$$

Where:

σ_0 = Shear stress = Force per unit area in flow direction;

γ_w = Specific weight of water;

D = Flow-depth;

S_f = Friction slope.

Particle movement can be further expressed at a specific point in a stream bed as incipient motion, which is the initial movement of a particle. The calculation of a critical shear stress or critical velocity can be performed at the threshold movement condition that assumes active hydraulic forces are equal to particle resistant forces. At the point of critical shear stress or critical velocity, a particle is just about to move. This means that values of shear stress or velocity greater than critical shear stress or critical velocity cause particles to be in motion, while particles will be at rest with values of shear stress and velocity lower than critical shear stress and velocity. An incipient motion calculation can provide an indication of erosion potential and stream stability. Fischenich (2001) provides a variation of the widely accepted and industry standard Shields equation for approximated critical shear stress considering different materials:

Clays:

$$\sigma_{cr} = 0.5d(\gamma_s - \gamma_w)\tan F$$

Silts & Sands:

$$\sigma_{cr} = 0.25d_0 - 0.6d(\gamma_s - \gamma_w)\tan F$$

Gravels & Cobbles:

$$\sigma_{cr} = 0.06d(\gamma_s - \gamma_w)\tan F$$

Where:

$d_0 = d\left[(S_g - 1)g\nu^{-2}\right]^{1/3}$;

σ_{cr} = Critical shear stress;

F = Soil grain angle of repose;

d = Soil diameter;

γ_s = Specific weight of sediment;

γ_w =Specific weight of water;

S_g = Sediment specific gravity;

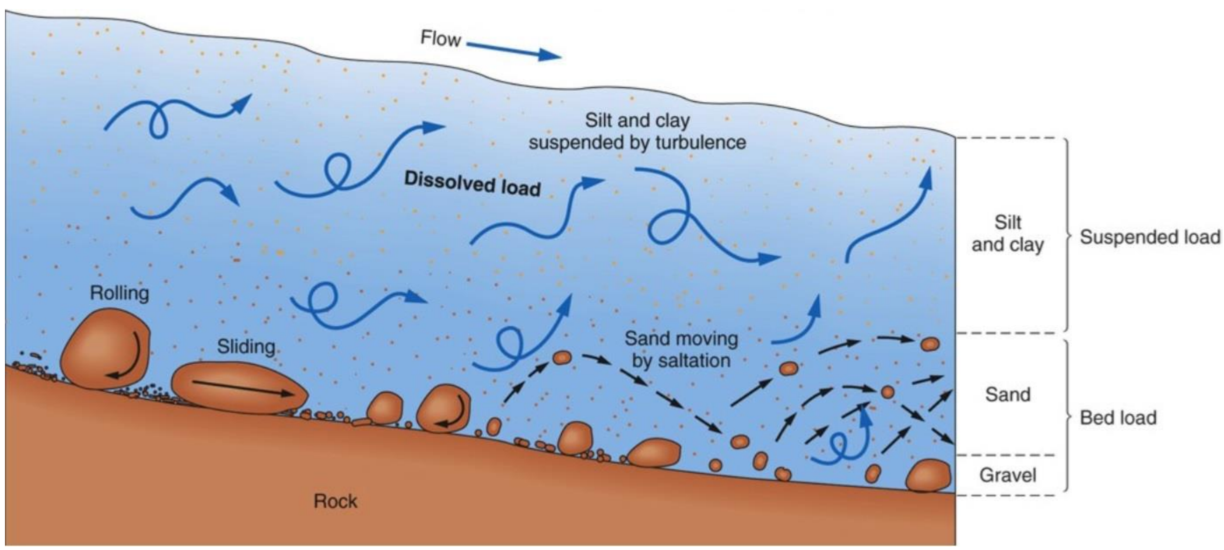
g = Gravity;

v = Water/sediment mixture kinematic viscosity.

The Shields equation and the beginning of motion is described in more detail in Index 3.5 of HDS No. 6.

Figure 872.3

Bed Load and Suspended Load



Modeling of a stream reach, although complex, can be performed in order to predict sediment transport potential on a larger scale, transport rates, volume, and capacity modeling. Several empirical sediment transport functions used in modeling have been developed and named after their creators, such as Einstein, Acker and White, Laursen-Copeland, Meyer-Peter Muller, and Yang. These functions are complex and notoriously data intensive. Three classic sediment transport formulae are discussed in detail in Index 4.5 of HDS No. 6 to illustrate sediment transport processes. While not often, resource agencies and flood control districts may request this type of analysis during the permit review process. If sediment modeling is necessary, HEC-RAS v4.1 (or higher), the Army Corps of Engineers' river and stream modeling software, contains sediment transport modeling capabilities using these transport functions and others.

Figure 872.4

Longitudinal Encroachments

Highway 49, North Fork Yuba River (Near Downieville) and Highway 190, Furnace Creek, (Death Valley)

(k) Stream Channel Form. Major factors affecting alluvial stream channel forms are:

- stream discharge, viscosity, temperature;
- sediment discharge;
- longitudinal slope;
- bank and bed resistance to flow;
- vegetation;
- geology, including types of sediments and;
- human activity.

At any location in a stream, the cross-sectional shape is dependent upon the volume flow-rate (flow), the composition of sediment transported through a section, and the integrity or gradation of the bed and bank materials. As water flows through the stream channel, it exerts a fluid shear stress on the bed and banks. For a constant and stable cross-sectional shape for a given flow at a specific location, the resisting bed and bank material shear stress must be equal to the fluid stress at every point in the stream cross section perimeter. In this state, a stream is in the threshold condition where each point along the perimeter is at the threshold of movement or incipient motion. This condition also indicates a dynamic equilibrium with scour and deposition of sediment being equal. As flow, velocity, and fluid shear stress increase, the amount of scour and sediment deposition will change, which will also change the stream cross section for a given bed/bank gradation.

The form and appearance of a stream can also be influenced by features within the stream profile, such as riffles and pools because of their effects on the acting fluid shear stress and velocity. Riffles are longitudinal sections of streams with higher velocity, where lower flow-depth usually caused by obstructions, such as gravels, cobbles, and boulders created by island or bar development. On the contrary, pools have higher flow-depth and lower velocity, and are typically comprised of finer silts and sands compared to a riffle. These bed materials associated with pools and riffles have an effect on resisting bed shear stress that will influence stream shape and stability. The alternating pool and riffle sequence is common for nearly all perennial streams that have gravel to boulder size bed formations. Different types of streams occur within a given subregion. Caltrans Hydromodification Requirements Guidance presents the various stream forms within each of the physiographic subregions of California, and is available from the Headquarters Office of Hydraulics and Stormwater Design.

(l) Floodplain Form. From a geomorphic perspective, floodplains are flatter lands adjacent to a river main channel that are dry until larger flows force water out of the stream channel into these overbank lands during significant flood events. Floodplains typically include the following features: the main stream channel itself, point islands or bars, oxbows and lakes, natural raised berms (levees) above floodplain surface, terraces, sloughs and depressions, overbank fine and coarse sediment deposition, scattered debris, and vegetation.

When water exceeds the capacity of the main channel, the conveyance of flow through the floodplain overbanks will differ from the main channel due to uniqueness of form (shape), gradient, alignment, and likely the flow resistance (roughness) of the floodplain versus the stream channel. Therefore, water will move and deposit varying sediment types differently, also at different frequency, creating a separate floodplain form. Once sediment is moved to the floodplain, coarser sediment is generally deposited along the streambanks forming levees, while finer sediment is dropped between the valley walls and the levees on the floodplain floor. Sediment is stored and becomes dormant until larger flows return to the floodplain that may convey the sediment down-valley.

Similar to the stream channel, floodplain form is directly linked to the sediment transport process, as well as floodplain stability affected by sediment supply and its movement. Fluid shear stress and velocity control the sediment/debris degradation and deposition properties within the floodplain that impact its form, landscape, and appearance. Because the floodplain can be dormant for considerable time depending on watershed hydrology, its form can remain relatively constant and preserved for extended periods, as well as be less dynamic than the stream channel.

(m) Streambank Erosion. Simply defined, streambank erosion occurs when the soil resisting strength is less than the driving forces acting on the bank. It can occur through bank-toe scour below the water line and bank mass failure from above. This erosion occurs first as a geotechnical failure followed by the hydraulic action that removes the failed soil and sediment by fluid shear stress. The hydraulic action further causes lateral scour of the bank and is the principal contributor to bank-toe failure. This is a natural process for both stable and unstable streams, but is exaggerated in the latter case. The degree of erosion can be influenced by impervious development in the watershed, agricultural use, and changes in climate. With or without these influences and whether a stream is stable or unstable, streambank erosion will take place at some level. Therefore, scour must be reduced at critical locations to protect highway structures and preserve public safety, although restraint needs to be exercised during the project development process so that a stream does not greatly change its morphology in response to the protection measures.

The driving and resisting forces for streambank erosion, mentioned above, are controlled by a series of factors. The factors that influence the calculation of the driving (active) forces within geotechnical failure are soil saturated unit weight, pore pressure, bank height, and angle of repose, as well as object surcharges within and above the bank such as vegetation. The effects of driving forces are commonly seen through soil saturation as a result of intense precipitation with subsequent increase in pore pressure and bank soil saturated unit weight that can cause mass bank failure. The forces that will resist and give soil its strength from geotechnical type failure are dependent upon effective soil cohesion, normal stress, pore pressure, and soil effective angle of friction.

During streambank erosion, the bank soil can fail by different modes. Generally speaking, steep slopes present slab-type or toppling failures where large slabs (blocks of soil) of the bank break away from the top and fall into the stream, while mild slopes show a rotational failure that begins at the bank toe causing soil to slide from above into the stream. Once the eroded soil reaches flowing water, it is usually transported downstream depending upon its size and composition.

As for bank-toe scour, its main influences are derived from bank soil composition and gradation, volume of sediment in transport, stream flow and stream gradient. These factors and the principles of scour and sediment movement from hydraulic forces are a reoccurring theme in fluvial geomorphology. The following paragraphs summarize the characteristics of unstable and stable banks;

(1) Unstable banks with moderate to high erosion rate occur when the slope angle of unstable banks typically exceed 30 percent, where a cover of woody vegetation is rarely present. At a bend, the point island or bar opposite of an unstable cut bank is likely to be bare at normal stage, but it may be covered with annual vegetation and low woody vegetation, particularly willows. Where very rapid erosion is occurring, the bankline may have irregular indentations. Fissures, which represent the boundaries of actual or potential slump blocks along the bankline indicate the potential for rapid bank erosion.

(2) Unstable banks with slow to moderate erosion rate occur when a bank is partly graded (smooth slope) and the degree of instability is difficult to assess where reliance is placed mainly on vegetation. The grading of a bank typically begins with the accumulation of slumped material at the base such that a slope is formed and progresses by smoothing of the slope and the establishment of vegetation.

(3) Stable banks with very slow erosion rate occur where banks tend to be graded to a smooth slope and the slope angle is usually less than about 30 percent. In most regions, the upper parts of stable banks are vegetated, but the lower part may be bare at normal stage, depending on bank height and flow regime of the stream. Where banks are low, dense vegetation may extend to the water's edge at normal stage. Mature trees on graded bank slopes are particularly convincing evidence for bank stability. Where banks are high, occasional slumps may occur on even the most stable graded banks. Shallow mountain streams that transport coarse bed sediment tend to have stable banks.

For a more detailed discussion of bank stability and the mechanics of bank failure see HEC 20.

(n) Young Valley. Typically young valleys are narrow V-shaped valleys with streams on steep gradients. Relief elevation greater than 1,000 ft is regarded as mountainous, while relief in the elevation range of 100 to 1,000 ft is regarded as hilly. Streams in mountainous regions are likely to have steep slopes, coarse bed materials (gravel or cobble-boulder), narrow floodplains, and have nonalluvial characteristics (i.e., supply-limited sediment transport rates). At flood stage, the stream flow covers all or most of the valley floor. The usual situation for such locations is a structure crossing a well-defined channel in which the design discharge will flow at a moderate to high velocity.

(1) Cross-Channel Location. A cross channel location is a highway crossing a stream on normal or skewed alignment. The erosive forces of parallel flow associated with a normal crossing are generally less of a threat than the impinging and eddy flows associated with a skewed crossing. The effect of constriction by projection of the roadway embankment into the channel should be assessed.

Characteristics to be considered include:

- Stream velocity.
- Scouring action of stream.
- Bank stability.
- Channel constrictions (artificial or natural).
- Nature of flow (tangential or curvilinear).
- Areas of impingement at various stages.
- Security of leading and trailing edges.

Common protection failures occur from:

- Undermining of the toe (inadequate depth/size of foundation), see Figure 872.5 and Table 872.2.
- Local erosion due to eddy currents.
- Inadequate upstream and downstream terminals or transitions to erosion-resistant banks or outcrops.
- Structural inadequacy at points of impingement overtopping.
- Inadequate rock size, see Table 872.2.
- Lack of proper gradation/ layering/ RSP fabric, leading to loss of embankment, see Table 872.2.

Any of the more substantial armor treatments can function properly in such exposures providing precautions are taken to alleviate the probable causes of failure. If the foundation is questionable for concreted-rock or other rigid types it would not be necessary to reject them from consideration but only to provide a more acceptable treatment of the foundation, such as heavy rock or sheet piling.

Whether the highway crosses a stream channel on a bridge or over a culvert, economic considerations often lead to constriction of the waterway. The most common constriction is in width, to shorten the structure. Next in frequency is obstruction by piers and bents of bridges or partitions of multiple culverts.

Table 872.2

Failure Modes and Effects Analysis for Riprap Revetment

Failure Modes	Effects on Other Components	Effects on Whole System	Detection Methods	Compensating Provisions
Translational slope or slump (slope failure)	Disruption of armor layer	Catastrophic failure	• Mound of rock at bank toe • Unprotected upper bank	• Reduce bank slope • Use more angular or smaller rock • Use granular filter rather than geotextile fabric
Particle erosion (rock undersized)	Loss of armor layer, erosion of filter	Progressive failure	• Rock moved downstream from original location • Exposure of filter	• Increase rock size • Modify rock gradation
Piping or erosion beneath armor (improper filter)	Displacement of armor layer	Progressive failure	• Scalloping of upper bank • Bank cutting • Void beneath and between rocks	• Use appropriate granular or geotextile filter
Loss of toe or key (under designed)	Displacement or disruption or armor layer	Catastrophic failure	• Slumping of rock • Unprotected upper bank	• Increase size, thickness, depth or extent of toe or key

Figure 872.5

Slope Failure Due to Loss of Toe

The risk of constricting the width of the waterway is closely related to the relative conveyance of the natural waterway obstructed, the channel scour, and to the channel migration. Constricting the width of flow at structures has the following effects:

- Increase in the upstream water surface elevation (backwater profile).
- Increase in flow velocity through the structure opening (waterway).
- Causes eddy currents around the upstream and downstream ends of the structure.

Unless protection is provided the eddy currents can erode the approach roadway embankment and the accelerated flow can cause scour at bridge abutments. The effects of erosion can be reduced by providing transitions from natural to constricted and back to natural sections, either by relatively short wingwalls or by relatively long training embankments or structures.

Channel changes, if properly designed, can improve conditions of a crossing by reducing skew and curvature and enlarging the main channel. Unfortunately, there are "side effects" which actually increase erosion potential. Velocity is almost always increased by the channel change, both by a reduction of channel roughness and increase of slope due to channel shortening. In addition, channel changes affecting stream gradient may have upstream and/or downstream effects as the stream adjusts in relation to its sediment load.

At crossing locations, lateral erosion can be controlled by positive protection, such as armor on the banks, rock spurs to deflect currents away from the banks, retards to reduce riparian velocity, or vertical walls or bulkheads. The life cycle cost of such devices should be considered in the economic studies to choose a bridge length which minimizes total cost.

Accurate estimates of anticipated scour depths are a prerequisite for safe, cost effective designs. Design criteria require that bridge foundations be placed below anticipated scour depths. For this reason, the design of protection to control scour at such locations is seldom necessary for new construction. However, if scour may undercut the toes of dikes or embankments positive methods including self-adjusting armor at the toe, jetties or retards to divert scouring currents away from the toe, or sill-shaped baffles interrupting transport of bedloads should be considered.

There is the potential for instability from saturated or inundated embankments at crossings with embankments projecting into the channel. Failures are usually reported as "washouts", but several distinct processes should be noted:

- Saturation of an embankment reduces its angle of repose. Granular fills with high permeability may "dissolve" steadily or slough progressively. Cohesive fills are less permeable, but failures have occurred during falling stages.
- As eddies carve scallops in the embankment, saturation can be accelerated and complete failure may be rapid. Partial or total losses can occur due to an upstream eddy, a downstream eddy, or both eddies eroding toward a central conjunction. Training devices or armor can be employed to prevent damage.
- If the fill is pervious and the pavement overtopped, the buoyant pressure under the slab will exceed the weight of slab and shallow overflow by the pressure head of the hydraulic drop at the shoulder line. A flat slab of thickness, t, will float when the upstream stage is 4t higher than the top of the slab. Thereafter the saturated fill usually fails rapidly by a combination of erosion and sloughing. This problem can occur or be increased when curbs, dikes, or emergency sandbags maintain a differential stage at the embankment shoulder. It is increased by an impervious or less pervious mass within the fill. Control of flotation, insofar as bank protection is concerned, should be obtained by using impervious armor on the upstream face of the embankment and a pervious armor on the downstream face.

  Culvert problem locations generally occur in and along the downstream transition. Sharp divergence of the high velocity flow develops outward components of velocity which attack the banks directly by impingement and indirectly by eddies entrained in quieter water. Downward components and the high velocity near the bed cause the scour at the end of the apron.

  Standard plans of warped wingwalls have been developed for a smooth transition from the culvert to a trapezoidal channel section. A rough revetment extension to the concrete wingwalls is often necessary to reduce high velocity to approximate natural flow. Energy dissipaters may be used to shorten the deceleration process when such a transition would be too long to be economical. Bank protection at the end of wingwalls is more cost effective in most cases.

(2) Parallel Location. With parallel locations the risk of erosion damage along young streams increases where valleys narrow and gradients steepen. The risk of erosion damage is greatest along the outer bend of natural meanders or where highway embankment encroaches on the main channel.

The *encroaching* parallel location is very common, especially for highways following mountain streams in narrow young valleys or canyons. Much of the roadway is supported on top of the bank or a berm and the outer embankment encroaches on the channel in a zone of low to moderate velocity. Channel banks are generally stable and protection, except at points of impingement, is seldom necessary.

The *constricting* parallel location is an extreme case of encroaching location, causing such impairment of channel that acceleration of the stream through the constriction increases its attack on the highway embankment requiring extra protection, or additional waterway must be provided by deepening or widening along the far bank of the stream.

In young valleys, streams are capable of high velocity flows during flood stages that may be damaging to adjacent highway facilities. Locating the highway to higher ground or solid support is always the preferred alternative when practical.

Characteristics to be considered include:

- High velocity flow.
- Narrow confined channels.
- Accentuated impingement.
- Swift overflow.
- Disturbed flow due to rock outcrops on the banks or within the main channel.
- Alterations in flow patterns due to the entrance of side streams into the main channel.

Protective methods that have proven effective are:

- Rock slope protection.
- Concreted-rock slope protection.
- Walls of masonry and concrete.
- Articulated concrete block revetments.
- Sacked concrete.

(o) Mature Valley. Typically mature valleys are broad V-shaped valleys with associated floodplains. See Figure 872.6. The gradient and velocity of the stream are low to moderate. Streams in regions of lower relief are usually alluvial and exhibit more problems because of lateral erosion in the channels. Vegetative cover, land use, and flow depth on the floodplain are also significant factors in stream channel stability. Changes in channel geometry with time are particularly significant during periods when alluvial channels are subjected to high flows, and few changes occur during relatively dry periods. Erosive forces during high-flow periods may have a capacity as much as 100 times greater than those forces acting during periods of intermediate and low-flow rates.

Figure 872.6

Mature Valley with Meandering Stream

Russian River near Geyserville

When considering the stability of alluvial streams, in most instances it can be shown that approximately 90 percent of all changes occur during that small percentage of the time when the flow equals or exceeds dominant discharge. A discussion of dominant discharge may be found in Hydraulic Design Series No. 6, but the bankfull flow condition is recommended for use where a detailed analysis of dominant discharge is not feasible. In addition to the general information previously given, the following applies to mature valleys:

(1) Cross-Channel Location. The usual situation is a structure crossing a braided or meandering normal flow channel. The marginal area subject to overflow is usually traversed by the highway on a raised embankment and may have long approaches extending from both banks.

Characteristics to be considered include:

- Shifting of the main channel.
- Skew of the stream to the structure.
- Foundation in deep alluvium.
- Erodible embankment materials.
- Channel constrictions, either artificial or natural, which may affect or control the future course of the stream.
- Variable flow characteristics at various stages.
- Stream acceleration at the structure.

Armor protection has proven effective to prevent erosion of road approach embankments, supplemented, if necessary, by stream training devices such as guide dikes, permeable retards or jetties to direct the stream through the structure. The abutments should not depend on the training dikes to protect them from erosion and scour. At bridge ends one of the more substantial armor types may be required, but bridge approach embankments affected only by overflow seldom require more than a light revetment, such as a thin layer of rocky material, vegetation, or a fencing along the toe of slope. For channel flow control upstream, the size and type of training system ranges from pile wings for high velocity, through permeable jetties for moderate velocity, to the earth dike suitable for low velocity.

The more common failures in this situation occur from:

- Lack of upstream control of channel alignment.
- Damage of unprotected embankments by overflow and return flow.
- Undercut foundations.
- Formation of eddies at abrupt changes in channel.
- Stranding of drift in the converging channel.

(2) Parallel Location. Parallel highways along mature rivers are often situated on or behind levees built, protected and maintained by other agencies. Along other streams, rather extensive protective measures may be required to control the action of these meandering streams.

Channel change is an important factor in locations parallel to mature streams. The channel change may be to close an embayment, to cut off an oxbow, or to shift the alignment of a long reach of a stream. In any case, positive means must be adopted to prevent the return of the stream to its natural course. For a straight channel, the upstream end is critical, usually requiring bank protection equivalent to the facing of a dam. On a curved channel change, all of the outer bend may be critical, requiring continuous protection. Continuous and resistive bank protection measures, such as riprap and longitudinal rock toes are primarily used to armor outer bends or areas with impinging flows. These continuous and concentrated high velocity areas will generally result in reduced aquatic habitat. Since streambank protection designs that consist of riprap, concrete, or other inert structures alone may be unacceptable for lack of environmental and aesthetic benefits. Resource agencies have increased interest in designs that combine vegetation and inert materials into living systems that can reduce erosion while providing environmental and aesthetic benefits.

(3) Desert Wash Locations. Particular consideration should be given to highway locations that traverse natural geographical features of desert washes, sand dunes, and other similar regions.

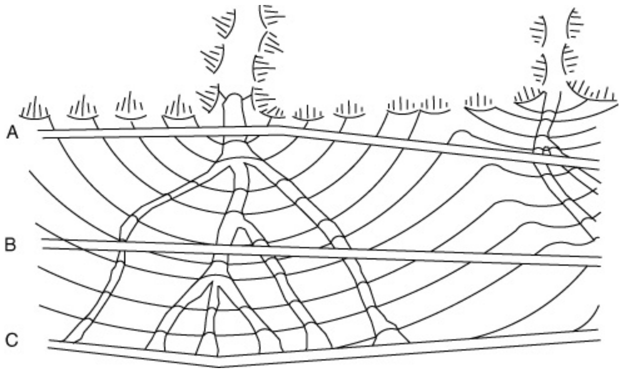
Desert washes are a prominent feature of the physiography of California. Many long stretches of highway are located across a succession of outwash cones. Infrequent discharge, often caused by flash floods, is typically wide and shallow, transporting large volumes of solids, both mineral and organic. Rather than bridge the natural channels, the generally accepted technique is to concentrate the flow by a series of guide dikes leading like a funnel to a relatively short crossing.

An important consideration at these locations is instability of the channel, see Figure 872.7. For a location at the top of a cone (Line A), discharge is maximum, but the single channel emerging from the uplands is usually stable. For a location at the bottom of the cone (Line C), instability is maximum with poor definition of the channel, but discharge is reduced by infiltration and stream dispersion. The energy of the stream is usually dissipated so that any protection required is minimal. The least desirable location is midway between top and bottom (Line B), where large discharge may approach the highway in any of several old channels or break out on a new line. Control may require dikes continuously from the top of the cone to such a mid-cone site with slope protection added near the highway where the converging flow is accelerated. See Figure 872.8, which depicts a typical alluvial fan.

Also common are roadway alignments which longitudinally encroach, or are fully within the desert wash floodplain, see Figure 872.9. Re-alignment to a stable location should be the first consideration, but restrictions imposed by federal or state agencies (National Park Service, USDA Forest Service, etc.) may preclude that option, somewhat similar to transverse crossings. The designer may need to consider allowing frequent overtopping and increased sediment removal maintenance since an “all weather design” within these regimes can often lead to large scale roadway washout.

Figure 872.7

Alternative Highway Locations Across Debris Cone



A. Cross at a single definite channel
B. A series of unstable indefinite channels and
C. A widely dispersed and diminished flow

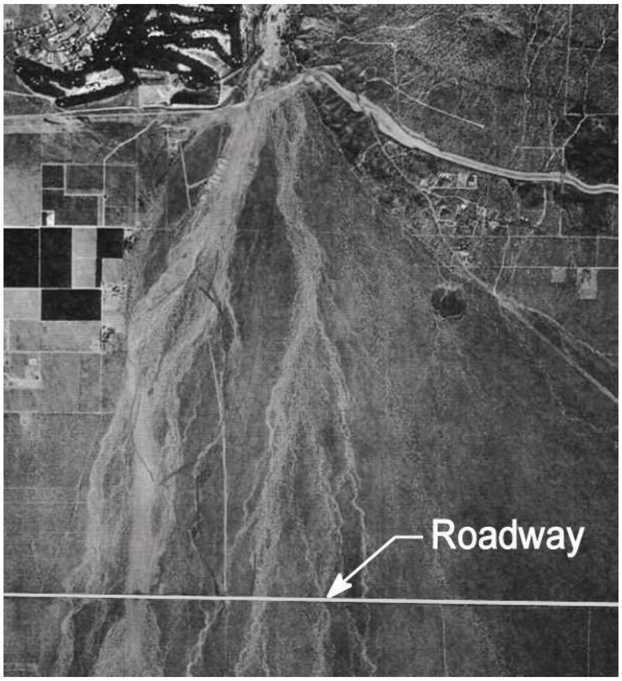
Characteristics to be considered include:

- The intensity of rainfall and subsequent run-off.
- The relatively large volumes of solids that are carried in such run-off.
- The lack of definition and permanence of the channel.
- The scour depths that can be anticipated.
- The lack of good foundation.

Effective protective methods include armor along the highway and at structures and the probable need for baffles to control the direction and velocity of flow. Installations of rock, fence, palisades, slope paving, and dikes have been successful.

Figure 872.8

Alluvial Fan



Typical multi-channel stream threads on alluvial fan.
Note location of roadway crossing unstable channels.

Figure 872.9

Desert Wash Longitudinal Encroachment

Road washout due to longitudinal location in desert wash channel

The Federal Emergency Management Agency (FEMA) Flood Hazard Mapping website contains information on recognizing alluvial fan landforms and methods for defining active and inactive areas. See their 2016 "Guidance for Flood Risk Analysis and Mapping; Alluvial Fans" at https://www.fema.gov/sites/default/files/2020-02/Alluvial_Fans_Guidance_Nov_2016.pdf.

(4) Construction, Easements, Access and Staging. A primary site consideration for any bank protection design is its constructability. This may include the need for supplemental plans and temporary construction easements for stage construction to accommodate equipment access. See Figure 872.10.

Figure 872.10

Stage Construction

(5) Biodiversity. The riparian area provides one of the richest habitats for large numbers of fish and wildlife species, which depend on it for food and shelter. Many species, including coho and Chinook salmon, steelhead, yellow-billed cuckoo, and the red-legged frog, are threatened or endangered in California. Natural riparian habitat also includes the assortment of native plants that occur adjacent to streams, creeks and rivers. These plants are well adapted to the dynamic and complex environment of streamside zones. A key threat to fish species in any migrating corridor therefore will include loss of riparian habitat and instream cover affecting juvenile rearing and outmigration.

For channel and habitat characterization and preliminary assessment relative to designing and obtaining project specific permits, District biologist staff should be consulted early on within the project planning phase for subject matter expertise regarding fisheries, habitat, and wildlife. District biologist staff can also perform an initial stream habitat assessment.

Numerous State and Federal agencies are responsible for fish management in California - including California Department of Fish and Wildlife, the National Marine Fisheries Service and the United Stated Army Corps of Engineers. Each agency has its own guidelines and jurisdiction. For example, detailed information on the requirements for fish habitat in riparian corridors may be found in Volume One and Two of the California Salmonid Stream Habitat Restoration Manual: https://wildlife.ca.gov/Grants/FRGP/Guidance.

872.4 Data Needs

The types and amount of data needed for planning and analysis of channel protection varies from project to project depending upon the class and extent of the proposed protection, site location environment, and geographic area. See Index 872.1. The data that is collected and developed including preliminary calculations, and alternatives considered should be documented in project development reports (Environmental Document, Project Report, etc.) or as a minimum in the project file. These records serve to guide the detailed designs, and provide reference background for analysis of environmental impacts and other needs such as permit applications and historical documentation for any litigation which may arise. See Index 873.3(3)(a)(2)(b) for rock sizing equation parameters.

Recommendations for data needs can be requested from the District Hydraulics Engineer or determined from Chapter 8 of FHWA's HDS No. 6, for a more complete discussion of data needs for highway crossings and encroachments on rivers. Further references to data needs are contained in Chapter 810, Hydrology and FHWA's HDS No. 2, Highway Hydrology and HEC 20, Stream Stability at Highway Structures.

872.5 Rapid Assessment

The National Pollutant Discharge Elimination System (NPDES) permit mandates a risk-based approach to be employed during planning and design for assessing stream stability at highway crossings. This approach involves conducting a rapid pre-project assessment of the vertical and lateral stability of the receiving stream channel related to an existing or planned highway crossing structure. If the rapid stability assessment (RSA) indicates potential problems, more detailed engineering analyses are required to determine if countermeasures are needed to stabilize the crossing to prevent the release of sediment. Therefore, if available, stream stability assessments for nearby highway crossings should be included in the site consideration for channel protection.

Section 3 of Caltrans Hydromodification Requirements Guidance Storm Water Best Management Practices Rapid Assessment of Stream Crossings Higher Level Stream Stability Analysis is an excellent resource for understanding the concepts of basic geomorphology and California earth science.

Table 8 of Assessing Stream Channel Stability at Bridges in Physiographic Regions (FHWA-HRT-05-072) presents an extensive listing of factors affecting stream stability.

Topic 873 – Design Concepts

873.1 Introduction

No attempt will be made here to describe in detail all of the various devices that have been used to protect embankments against scour. Methods and devices not described may be used when justified by economic analysis. Not all publicized treatments are necessarily suited to existing conditions for a specific project.

A set of plans and specifications must be prepared to define and describe the protection that the design engineer has in mind. These plans should show controlling factors and an end product in such detail that there will be no dispute between the construction engineer and contractor. To serve the dual objectives of adequacy and economy, plans and specifications should be precise in defining materials to be incorporated in the work, and flexible in describing methods of construction or conformance of the end product to working lines and grades.

Recommendations on channel lining, slope protection, and erosion control materials can be requested from the District Hydraulic Engineer, the District Materials Branch and the Office of Highway Drainage and Water Quality Design in Headquarters. The District Landscape Architect will provide recommendations for temporary and permanent erosion and sediment control measures. The Caltrans Bank and Shore Protection Committee is available on request to provide advice on extraordinary situations or problems and to provide evaluation and formal approvals for acceptable non-standard designs. See Index 802.3 for further information on the organization and functions of the Committee.

Combinations of armor-type protection can be used, the slope revetment being of one type and the foundation treatment of another. The use of rigid, non-flexible slope revetment may require a flexible, self-adjusting foundation for example: concreted-rock on the slope with heavy rock foundation below, or PCC slope paving with a steel sheet-pile cutoff wall for foundation.

Bank protection may be damaged while serving its primary purpose. Lower cost replaceable facilities may be more economical than expensive permanent structures. However, an expensive structure may be economically warranted for highways carrying large volumes of traffic or for which no detour is available.

Cost of stone is extremely sensitive to location. Variables are length of haul, efficiency of the quarry in producing acceptable sizes, royalty to quarry and, necessity for stockpiling and rehandling. On some projects the stone may be available in roadway excavation.

873.2 Design High Water and Hydraulics

The most important, and often the most perplexing obligation, in the design of bank and shore protection features is the determination of the appropriate design high water elevation to be used. The design flood stage elevation should be chosen that best satisfies site conditions and level of risk associated with the encroachment. The basis for determining the design frequency, velocity, backwater, and other limiting factors should include an evaluation of the consequences of failure on the highway facility and adjacent property. Stream stability and sediment transport of a watercourse are critical factors in the evaluation process that should be carefully weighted and documented. Designs should not be based on an arbitrary storm or flood frequency.

A suggested starting point of reference for the determination of the design high water level is that the protection withstands high water levels caused by meteorological conditions having a recurrence interval of one-half the service life of the protected facility. For example, a modern highway embankment can reasonably be expected to have a service life of 100 years or more. It would therefore be appropriate to base the preliminary evaluation on a high water elevation resulting from a storm or flood with a 2 percent probability of exceedance (50 year frequency of recurrence). The first evaluation may have to be adjusted, either up or down, to conform with a subsequent analysis which considers the importance of the encroachment and level of related risks which may include consideration of historic high water marks and climate change. Scour countermeasures protecting structures designed by the Division of Engineering Services (DES) may include consideration of floods greater than a 1 percent probability of exceedance (100 year frequency of recurrence).

There is always some risk associated with the design of protection features. Special attention must be given to life threatening risks such as those associated with floodplain encroachments. Significant floodplain risks are classified as those having probability of:

- Catastrophic failure with loss of life.
- Disruption of fire and ambulance services or closing of the only evacuation route available to a community.

Refer to Topic 804, Floodplain Encroachments, for further discussion on evaluation of risks and impacts.

(1) *Streambank Locations.* The velocity along the banks of watercourses with smooth or uniformly rough tangent reaches may only be a small percentage of the average stream velocity. However, local irregularities of the bank and streambed may cause turbulence that can result in the bank velocity being greater than that of the central thread of the stream. The location of these irregularities is not always permanent as they may be caused by local scour, deposition of rock and sand, or stranding of drift during high water changes. It is rarely economical to protect against all possibilities and therefore some damage should always be anticipated during high water stages.

Essential to the design of streambank protection is sufficient information on the characteristics of the watercourse under consideration. For proper analysis, information on the following types of watercourse characteristics must be developed or obtained:

- Design Discharge
- Design High Water Level
- Flow Types
- Channel Geometry
- Flow Resistance
- Sediment Transport

Refer to Chapter 810, Hydrology, for a general discussion on hydrologic analysis and specifically to Topic 817, Flood Magnitudes; Topic 818, Flood Probability and Frequency; and Topic 819, Estimating Design Discharge. For a detailed discussion on the fundamentals of alluvial channel flow, refer to Chapter 3, HDS No. 6, and to Chapter 4, HDS No. 6, for further information on sediment transport.

(2) *Ocean & Lake Shore Locations.* Refer to Chapter 880 for information needed to design shore protection.

873.3 Armor Protection

(1) *General.* Armor is the artificial surfacing of bed, banks, shore or embankment to resist erosion or scour. Armor devices can be flexible (self-adjusting) or rigid.

Hard armoring of stream banks, primarily with rock slope protection (RSP), has been the most common means of providing long-term protection for transportation facilities, and most importantly, the traveling public. With many years of use, dozens of formal studies and thousands of constructed sites, RSP is the armor type for which there exists the most quantifiable data on performance, constructability, maintainability and durability, and for which there exist several nationally recognized design methods.

Due to the above factors, RSP is the general standard against which other forms of armoring are compared.

The results of internal research led to the publication of Report No. FHWA-CA-TL-95-10, “California Bank and Shore Rock Slope Protection Design”. Within that report, the methodology for RSP design adopted as the Departmental standard for many years, was the California Bank and Shore, (CaBS), layered design. The CaBS layered design methodology and its associated gradations have become obsolete.

FHWA Hydraulic Engineering Circular No. 23 (HEC 23) presents guidelines for RSP for a range of applications, including: RSP on streams and river banks, bridge piers and abutments, and bridge scour countermeasures such as guide banks and spurs. These guidelines were formally adopted by the Caltrans Bank and Shore Protection Committee with a modified version of HEC 23 gradations. See Tables 873.3A and 873.3B as well as HEC 23, Volume 1, Chapter 5 and Design Guideline 4, 5, 11, 12, 15 and 16 from Volume 2. Section 72 of the Standard Specifications provides all construction and material specifications for RSP designs. While standards (i.e., Standard Plans, Standard Specifications and/or SSP’s) do exist for some other products discussed in this Chapter (most notably for gabions, but also for certain rolled or mat-style erosion control products), their primary application is for relatively flat slope or shallow ditch erosion control (gabions are also used as an earth retaining structure, see Topic 210 for more details).

Rigid and other armor types listed below are viable and may be considered where conditions warrant. Although the additional step of headquarters approval of any nonstandard designs is required, designers are encouraged to consider alternative designs, particularly those that incorporate vegetation or products naturally present in stream environments. The District Landscape Architect can provide design assistance together with specifications and details for the vegetative portion of this work.

(a) Flexible Types.

- Rock slope protection.
- Gabions, Standard Plan D100A and D100B.
- Precast concrete articulated blocks.

(b) Rigid Types.

- Concreted-rock slope protection.
- Partially-grouted rock slope protection.
- Sacked concrete slope protection.
- Concrete filled cellular mats.

(2) *Bulkheads.* The bulkhead types are steep or vertical structures, like retaining walls, that support natural slopes or constructed embankments which include the following:

- Gravity or pile supported concrete or masonry walls.
- Sheet piling

(a) General Design Criteria. In selecting the type of flexible or rigid armor protection to use the following characteristics are important design considerations.

(1) The lower limit, or toe, of armor should be below anticipated scour or on bedrock. If for any reason this is not economically feasible, a reasonable degree of security can be obtained by placement of additional quantities of heavy rock at the toe which can settle vertically as scour occurs.

(2) In the case of slope paving or any expensive revetment which might be seriously damaged by overtopping and subsequent erosion of underlying embankment, extension above design high water may be warranted. The usual limit of extension for streambank protection above design high water is 1 foot to 2 feet in unconstricted reaches and 2 feet to 3 feet in constricted reaches.

(3) The upstream terminal can be determined best by observation of existing conditions and/or by measuring velocities along the bank. The terminal should be located to conform to outcroppings of erosion-resistant materials, trees, shrubs or other indications of stability.

In general, the upstream terminal on bends in the stream will be some distance upstream from the point of impingement or the beginning of curve where the effect of erosion is no longer damaging.

(4) When possible, the downstream terminal should be made downstream from the end of the curve and against outcroppings, erosion-resistant materials, or returned securely into the bank so as to prevent erosion by eddy currents and velocity changes occurring in the transition length.

(5) The encroachment of embankment into the stream channel must be considered with respect to its effect on the conveyance of the stream and possible damaging effect on properties upstream due to backwater and downstream due to increased stream velocity or redirected stream flow.

(6) A smooth surface will generally accelerate velocity along the bank, requiring additional treatment (e.g., extended transition, cut-off wall, etc.) at the downstream terminal. Rougher surfaces tend to keep the thread of the stream toward the center of the channel.

(7) Heavy-duty armor used in exposures along the ocean shore may be influenced or dictated by economics, or the feasibility of handling heavy individual units.

(3) Flexible Revetments.

(a) Streambank Rock Slope Protection.

(1) General Features. This kind of protection, commonly called riprap, consists of rock courses placed upon the embankment or the natural slope along a stream. Rock, as a slope protection material, has a number of desirable features which have led to its widespread application.

It is usually the most economical type of revetment where stones of sufficient size and quality are available, it also has the following advantages:

- It is flexible and is not impaired nor weakened by slight movement of the embankment resulting from settlement or other minor adjustments.
- Local damage or loss is easily repaired by the addition of similar sized rock where required.
- Construction is not complicated and special equipment or construction practices are not usually necessary. (Note that Method A placement of very large rock may require large cranes or equipment with special lifting capabilities).
- Appearance is natural, and usually acceptable in recreational and scenic areas.
- If exposed to fresh water, vegetation may be induced to grow through the rocks adding structural value to the embankment material and restoring natural roughness. See Index 873.3(3)(a)(2)(d) for further vegetative rock slope protection information.

- Additional thickness (i.e., mounded toe design) can be provided at the toe to offset possible scour when it is not feasible to found it upon bedrock or below anticipated scour.
- It is salvageable, may be stockpiled and reused if necessary.

In designing the rock slope protection for a given embankment the following determinations are to be made for the typical section.

- Depth at which the stones are founded (bottom of toe trench).
- Elevation at the top of protection.
- Thickness of protection.
- Need for geotextile or rock filter material.
- Face slope.
- Need for and location of plant tubes.

(a) *Placement.* Two different methods of placement for rock slope protection are allowed under Section 72 of the Standard Specifications: Placement under Method A requires considerable care, judgment, and precision and is consequently more expensive than Method B. Method A should be specified primarily where large rock is required, but also for relatively steeper slopes.

(b) *Foundation Treatment.* The foundation excavation must afford a stable base on bedrock or extend below anticipated scour.

Terminals of revetments are often destroyed by eddy currents and other turbulence because of nonconformance with natural banks. Terminals should be secured by transitions to stable bank formations, or the end of the revetment should be reinforced by returns of thickened edges.

While a significant amount of research is currently being conducted, few methods exist for estimating scour along stream banks. One of the few is the method contained in HEC 23 Volume 1, Index 4.3.5 and the CHANLPRO Program developed by the U.S.

Army Corps of Engineers. Based on the flume studies at the Corps' Waterways Experiment Station, the program is primarily used by the Corps for RSP designs on streams with 2 percent or lesser gradients, but contains an option for scour depth estimates in bends for sand channels. CHANLPRO is available at the following USACE website: https://apps.dtic.mil/dtic/tr/fulltext/u2/a351838.pdf along with a user guide containing equations, charts, assumptions and limitations to the method and example problems.

(c) *Embankment Considerations.* Embankment material is not normally carried out over the rock slope protection so that the rock becomes part of the fill. With this type of construction fill material can filter down through the voids of the large stones and that portion of the fill above the rocks could be lost. If it is necessary to carry embankment material out over the rock slope protection a geotextile is required to prevent the loses of fill material.

The embankment fill slope is usually determined from other considerations such as the angle of repose for embankment material, or the normal 1V:4H specified for high-standard roads. If the necessary size of rock for the given exposure is not locally available, consideration should be given to flattening of the embankment slope to allow a smaller size stone, or substitution of other types of protection. On high embankments, alternate sections on several slopes should be compared, practically and economically; flatter slopes require smaller stones in thinner sections, but at the expense of longer slopes, a lower toe elevation, increased embankment, and perhaps additional right of way.

Where the roadway alignment is fixed, slope flattening will often increase embankment encroachment into the stream. When such an encroachment is environmentally or technically undesirable, the designer should consider various vertical, or near vertical, wall type alternatives to provide adequate stream width, allowing natural channel migration and the opportunity for enhancing habitat.

(d) *Rock Slope Protection Fabric.* Rock Slope Protection fabrics are described in Standard Specification Section 96. The RSP fabric placement ensures that fine soil particles do not migrate through the RSP due to hydrostatic forces and, thus, eliminate the potential for bank failure. The use of RSP fabric provides an inexpensive layer of protection retaining embankment fines in lieu of placing a gravel filter of small, well graded materials. See Index 873.3(3)(a)(1)(e) "Gravel Filter."

Stronger and heavier RSP fabrics than those listed in the Standard Specifications are manufactured. They are used in special designs for larger than standard RSP sizes, or emergency installations where placement of large RSP must be placed directly on the fabric. These heavy weight fabrics have unit weights of up to 16 ounces per square yard. Contact the Headquarters Hydraulic Engineer for assistance regarding usage applications of heavy weight RSP fabrics.

(e) *Gravel Filter*. Generally, RSP fabric should always be used unless there is a permit requirement that precludes the placement of fabric. Where RSP fabric cannot be placed, such as in stream environments where CA Fish & Wildlife and NOAA Fisheries strongly discourage the use of RSP Fabric, a gravel filter is usually necessary with most native soil conditions to stop fines from bleeding through the typical RSP classes. A gravel filter will be specified and placed between the native base soil and RSP for hybrid revetments to avoid conflicts associated with planting vegetation and placing RSP fabric together. A universal gravel filter gradation is presented in Design Information Bulletin No. 87 (see Table H, Index 7.1.2), which should work for many stream sites in California and eliminate the need for a site-specific gravel filter design for every project.

When a gravel filter is to be placed, the designer is advised to work with the District Materials Office to get a recommendation for the necessary gradation to work effectively with both the native backfill and the base layer of the RSP that is being placed. Among the methods available for designing the gravel filter are the Terzaghi method, developed exclusively for situations where the native backfill is sand, and the Cisten-Ziems method, which is often used for a broad variety of soil types and recommended in HEC 23. Where streambanks must be significantly rebuilt and reconfigured with imported material before RSP placement, the designer must ensure that the imported material will not bleed through the designed gravel filter. See HEC 23 Volume 2, Design Guideline 16, Index 16.2.1 Granular Filter Design Procedure and 16.3.1 Granular filter (design example).

(2) Streambank Protection Design. In the lower reaches of larger rivers wave action resulting from navigation or wind blowing over long reaches may be much more serious than velocity. A 2 foot wave, for example, is more damaging than direct impingement of a current flowing at 10 feet per second. Therefore, consideration of a wave attack based design may be necessary. See Chapter 880 for further information.

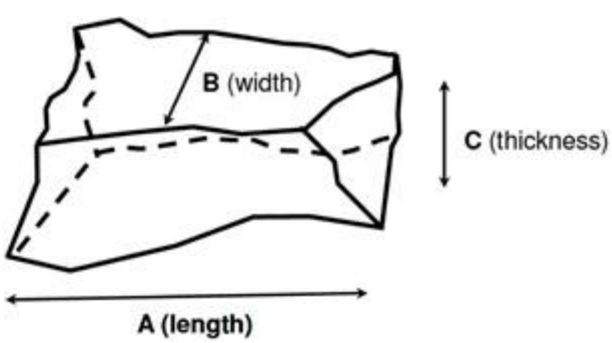
Well designed streambank rock slope protection should:

- Assure stability and compatibility of the protected bank as an integral part of the channel as a whole.
- Connect to natural bank, bridge abutments or adjoining improvements with transitions designed to ease differentials in alignment, grade, slope and roughness of banks.
- Eliminate or ease local embayments and capes so as to streamline the protected bank.
- Consider the effects of backwater above constrictions, superelevations on bends, as well as tolerance of occasional overtopping.
- Not be placed on a slope steeper than 1.5H:1V. Flatter slopes use lighter stones in a thinner section and encourage overgrowth of vegetation, but may not be permissible in narrow channels.
- Use stone of adequate weight to resist erosion, derived from Index 873.3(3)(a)(2)(b).
- Prevent loss of bank materials through interstitial spaces of the revetment. Rock slope protection fabric should be used.
- Rest on a good foundation on bedrock or extend below the depth of probable scour. If questionable, use heavy bed stones and provide a wide base section with a reserve of material to slough into local scour holes (i.e., mounded toe).
- Reinforce critical zones on outer bends subject to impinging flow, using heavier stones, thicker section, and deeper toe.
- Be constructed of rock of such shape as to form a stable protection structure of the required section. Rounded boulders or cobbles must not be used on prepared ground surfaces having slopes steeper than 2.5H:1V.

(a) *Stone Shape.* The shape of a stone can be generally described by designating three axes of measurement: major, intermediate, and minor, also known as the "A, B, and C" axes, as shown in Figure 873.3A.

Figure 873.3A

Stone Shape



Riprap stones should not be thin and platy, nor should they be long and needle-like. Therefore, specifying a maximum allowable value of the ratio A/C, also known as the shape factor, provides a suitable measure of particle shape, since the B axis is intermediate between the two extremes of length A and thickness C. A maximum allowable value for A/C of 3.0 is recommended.

Based on field studies, the recommended relationship between stone size and weight is given by:

$$W = 0.85(\gamma_s d^3)$$

Where:

W = Weight of stone, lb;

d = Size of intermediate ("B") axis, ft;

γ_s = Density of stone, lb/ft^3;

= S_g γ_w

Where:

γ_w = 62.4 lb/ft^3;

S_g = Specific gravity of stone.

Tables 873.3A and 873.3B provide recommended gradations for eleven standard classes of riprap based on median particle size d_{50} as determined by the dimension of the intermediate ("B") axis. The d or W refers to size or weight, respectively. The number is the percent finer by weight. Tables 873.3A and 873.3B are modified versions of Tables 4.1 and 4.2 in HEC 23, Volume 2, Design Guideline 4, which provide recommended gradations for ten standard classes of riprap and conform to those recommended in NCHRP Report 568 (Lagasse et al. 2006). The gradation criteria in Table 873.3A are based on a nominal or "target" d_{50}. See Index 873.3(3)(a)(2)(b) for equations to calculate d_{30} and d_{50}. The most significant modifications to Tables 873.3A and 873.3B from the gradations shown in Tables 4.1 and 4.2 are to the d_{100max} and W_{100max} gradation for classes VIII through XI, which have been truncated for practicality. An additional class XI is included in Tables 873.3A and 873.3B. Contact the Headquarters Hydraulic Engineer if more information is needed on the modification to the HEC 23 gradations.

Based on the recommended relationship between size and weight, which assumes the volume of the stone is 85% of a cube, Table 873.3B provides the equivalent particle weights for the same eleven classes as Table 873.3A using a specific gravity of 2.65 for the particle density.

Table 873.3A

RSP Class by Median Particle Size[3]

Nominal RSP Class by Median Particle Size[3]		d_{15} (in)		d_{50} (in)		d_{100} (in)	Placement Method
Class [1], [2]	Size (in)	Min	Max	Min	Max	Max	
I	6	3.7	5.2	5.7	6.9	12.0	B
II	9	5.5	7.8	8.5	10.5	18.0	B
III	12	7.3	10.5	11.5	14.0	24.0	B
IV	15	9.2	13.0	14.5	17.5	30.0	B
V	18	11.0	15.5	17.0	20.5	36.0	B
VI	21	13.0	18.5	20.0	24.0	42.0	A or B
VII	24	14.5	21.0	23.0	27.5	48.0	A or B
VIII	30	18.5	26.0	28.5	34.5	48.0	A or B
IX	36	22.0	31.5	34.0	41.5	52.8	A
X	42	25.5	36.5	40.0	48.5	60.5	A
XI	46	28.0	39.4	43.7	53.1	66.6	A

NOTES:

[1]Rock grading and quality requirements per Standard Specifications.

[2]RSP-fabric Type of geotextile and quality requirements per Section 96 Rock Slope Protection Fabric of the Standard Specifications. For RSP Classes I thru VIII, use Class 8 RSP-fabric which has lower weight per unit area and it also has lower toughness (tensile x elongation, both at break) than Class 10 RSP-fabric. For RSP Classes IX thru XI, use Class 10 RSP-fabric.

[3]Intermediate, or B dimension (i.e., width) where A dimension is length, and C dimension is thickness.

Table 873.3B

RSP Class by Median Particle Weight[3]

Nominal RSP Class by Median Particle Weight		W_{15} (lb)		W_{50} (lb)		W_{100} (lb)	Placement Method
Class [1], [2]	Weight	Min	Max	Min	Max	Max	
I	20 lb	4	11	15	27	140	B
II	60 lb	14	39	50	94	470	B
III	150 lb	32	94	120	220	1,100	B
IV	300 lb	63	180	250	440	2,200	B
V	1/4 ton	110	300	400	700	3,800	B
VI	3/8 ton	180	520	650	1,100	6,000	A or B
VII	1/2 ton	250	750	1000	1,700	9,000	A or B
VIII	1 ton	520	1,450	1,900	3,300	9,000	A or B
IX	2 ton	870	2,500	3,200	5,800	12,000	A
X	3 ton	1,350	4,000	5,200	9,300	18,000	A
XI	4 ton	1,800	5,000	6,800	12,200	24,000	A

NOTES:

*(1)*Rock grading and quality requirements per Standard Specifications.

*(2)*RSP-fabric Type of geotextile and quality requirements per Section 96 Rock Slope Protection Fabric of the Standard Specifications. For RSP Classes I thru VIII, use Class 8 RSP-fabric which has lower weight per unit area and it also has lower toughness (tensile x elongation, both at break) than Class 10 RSP-fabric. For RSP Classes IX thru XI, use Class 10 RSP-fabric.

*(3)*Values shown are based on Table 873.3A dimensions and an assumed specific gravity of 2.65. Weight will vary based on density of rock available for the project.

(b) *Stone Size*. Where stream velocity governs, rock size may be estimated from the following formula, which can be used with uniform or gradually varying flow. Coefficients are included to account for the desired safety factor for design, specific gravity of the riprap stone, bank slope, and bendway character;

$$d_{30} = y(S_f C_S C_V C_T)\left[\frac{V_{des}}{\sqrt{K_1(S_g - 1)gy}}\right]^{2.5}$$

Where:

d_{30} =Particle size for which 30% is finer by weight, ft;

y = Local depth of flow, ft;

S_f = Safety factor (typically = 1.1);

C_S =Stability coefficient (for blanket thickness $1.5d_{50}$ or d_{100}, whichever is greater) = 0.30 for angular rock;

C_V =Velocity distribution coefficient;

=1.0 for straight channels or the inside of bends;

=1.283 – 0.2 log (R_c/W) for the outside of bends (1.0 for R_c/W > 26);

=1.25 downstream from concrete channels;

=1.25 at the end of dikes;

C_T =Blanket thickness coefficient = 1.0;

S_g =Specific gravity of stone (2.5 minimum);

g =Acceleration due to gravity, 32.2 ft/s^2;

V_{des} =Characteristic velocity for design, defined as the depth-averaged velocity at a point 20% upslope from the toe of the revetment, ft/s;

For natural channels,

$V_{des}=V_{avg}$ (1.74 – 0.52 log (R_c/W))

$V_{des}=V_{avg}$ for R_c/W > 26

For trapezoidal channels,

$V_{des}=V_{avg}$ (1.71 – 0.78 log (R_c/W))

$V_{des}=V_{avg}$ for R_c/W > 8

Where:

R_c = Centerline radius of curvature of channel bend, ft;

W = Width of water surface at upstream end of channel bend, ft;

V_{avg} = Channel cross-sectional average velocity, ft/s;

K_1 = Side slope correction factor;

$$K_1 = \sqrt{1 - \left[\frac{\sin(\theta - 14°)}{\sin 32°}\right]^{1.6}}$$

Where:

θ =is the bank angle in degrees.

The flow depth "y" used in the above equation is defined as the local flow depth. The flow depth at the toe of slope is typically used for bank revetment applications; alternatively, the average channel depth can be used. The smaller of these values will result in a slightly larger computed d_{30} size, since riprap size is inversely proportional to ($y^{0.25}$). The blanket thickness coefficient (C_T) is 1.0 for standard riprap applications where the thickness is equal to $1.5d_{50}$ or d_{100}, whichever is greater. Because limited data is available for selecting lower values of C_T when greater thicknesses of riprap are used, a value of 1.0 is reasonable for all applications. The recommended Safety Factor S_f is 1.1 for bank revetment. Greater values should be considered where there is significant potential for ice or impact from large debris, freeze-thaw degradation that would significantly decrease particle size, or large uncertainty in the design variables, especially velocity. The specific gravity (S_g) of stone is commonly taken as 2.65 for planning purposes, however, this will result in a less conservative design than utilizing a 2.5 specific gravity assumption, which would be the minimum accepted in the field. Therefore, the designer should contact the District Materials Engineer in the project's area and determine if there is any history of RSP materials used in that region. Where such information or history is unavailable, use of a 2.5 specific gravity within the design should be considered.

The d_{30} size of the riprap is related to the recommended median (d_{50}) size by:

$d_{50} = 1.20d_{30}$

Using standard sizes the appropriate gradation can be achieved by selecting the next larger size class, thereby creating a slightly over-designed structure, but economically a less expensive one. For example, if a riprap sizing calculation results in a required d_{50} of 16.8 inches, Class V riprap should be specified because it has a nominal d_{50} of 18 inches. See Table 873.3A.

A limitation to the rock size equation above is that the longitudinal slope of the channel should not be steeper than 2.0% (0.02 ft/ft). For steeper channels, the riprap sizing approach for overtopping flows presented in HEC 23, Volume 2, Design Guideline 5 should be considered and the results compared with the rock size equation above .

Where wave action is dominant, design of rock slope protection should proceed as described for shore protection, see Chapter 880.

(c) *Design Height*. The top of rock slope protection along a stream bank should be carried to the elevation of the design high water plus some allowance for freeboard. Cost and severity of damage if overtopped as well as the importance of the facility should also be considered. The goal for the design high water is based on the 50-year (2% probability) flow, but can be modified using engineering judgment which may include consideration of historic high water marks and climate change. This stage may be exceeded during infrequent floods, usually with little or no damage to the upper slope. See Hybrid RSP cross section in Figure 873.3D for an example showing the top of rock slope protection.

When determining freeboard, or the height above design high water from which the RSP is to extend, one should consider: the size and nature of debris in the flow; the resulting potential for damage to the bank, the potential for streambed aggradation; and the confidence in data used to estimate design highwater. Freeboard may also be affected by regulatory or local agency requirements. Freeboard may be more generous on the outside bends of channels, or around critical bridges.

The 50-year design high water plus freeboard goal should be followed whenever possible, but the biggest exception to this goal occurs when the design height exceeds the main channel top of bank. Because floodplain overbank areas can be wide and extensive, the footprint of the RSP could grow exponentially if extended above and beyond the top of bank. This increased footprint would bring higher costs and permitting challenges that could make a project no longer viable. Given this possibility, the RSP vertical limit (height) should typically end at the main channel top of bank; however, a vegetation component may extend above and beyond the top of bank.

For cases where significant erosion has occurred above the main channel top of bank into its overbank(s), contact the District Hydraulic Engineer to discuss alternatives for repair and protection.

Design Example – The following example reflects the HEC 23 method for designing RSP. The designer is encouraged to review Design Guideline 4, Riprap Revetment from HEC 23, Volume 2. The following example assumes that the designer has conducted the appropriate site assessments and resulting calculations to establish average stream velocity, flow depth at bank toe, estimated depth of scour, stream alignment (i.e., parallel or impinging flow), width of channel, radius of bend (if impinging flow), length and side slope 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 of 9.8 feet per second
- Flow depth of 11.4 feet at bank toe
- Estimated scour depth – 3.5 feet
- Length of bank requiring protection – 550 feet
- Bank slope – 2:1
- Specific gravity of rock used for RSP – 2.54 (based on data from local quarry)
- Embankment is on outside of stream bend of 100 ft wide natural channel on a bend that has a centerline radius (R_c) of 500 ft. The radius of curvature divided by width (R_c/W) is 5.0.
- A desired factor of safety (Sf) of 1.2.

Determine the target d_{50}, select appropriate RSP class from Table 873.3A and determine the blanket thickness:

Step 1: Compute the side slope correction factor:

$$K_1 = \sqrt{1 - \left(\frac{\sin(\theta - 14°)}{\sin 32°}\right)^{1.6}}$$

$$= \sqrt{1 - \left(\frac{\sin(26.6° - 14°)}{\sin 32°}\right)^{1.6}}$$

$$= 0.87$$

Step 2: Select the appropriate stability coefficient for riprap: C_s (for blanket thickness $1.5d_{50}$ or d_{100}, whichever is greater) = 0.30 for angular rock

Step 3: Compute the vertical velocity factor (C_v) for R_c/W = 5.0:
C_v = 1.283 − 0.2 log (R_c/W)
= 1.283 − 0.2 log (5.0)
= 1.14

Step 4: Compute local velocity on the side slope (V_{des}) for a natural channel with R_c/W = 5.0:
V_{des} = V_{avg} [1.74− 0.52 log (R_c/W)]
= 9.8[1.74 − 0.52 log (5.0)]
= 13.5 ft/s

Step 5: Compute the d_{30} size using stone size equation from Index 873.2(2)(a)(2)(b):

$$d_{30} = S_f C_s C_v y \left[\frac{V_{des}}{\sqrt{(Sg - 1)K_1 g y}}\right]^{2.5}$$

$$= (1.2)(0.3)(1.14)(11.4) \times \left[\frac{13.5}{\sqrt{(2.54 - 1)(0.87)(32.2)(11.4)}}\right]^{2.5}$$

= 1.35 ft

Step 6: Compute the d_{50} size = $1.2d_{30}$ = 1.2(1.35)
= 1.62 ft = 19 inches.

Note: Use next larger size class (see Table 873.3A)

Step 7: Select Class VI riprap from Table 873.3A: d_{50} = 21 inches

Step 8: Blanket thickness = $1.5d_{50}$ or d_{100}, whichever is greater
$1.5d_{50}$ = 1.5 (21 inches)
= 31.5 inches
d_{100} = 42 inches, therefore, use 42 inches

Step 9: Determine the depth of riprap embedment below the streambed at the toe of the bank slope:

Since toe scour is expected to be 3.5 ft, the 2H:1V slope should be extended below the ambient bed level 7 ft horizontally out from the toe to accommodate this scour. Alternatively, a mounded riprap toe 3.5 ft high could be established at the base of the slope and allowed to self-launch when toe scour occurs, see Figure 873.3D.

Step 10: Assess Stream Impact Due to Revetment. In some cases, the thickness of the completed RSP revetment creates a narrowing of the available stream channel width, to the extent that stream velocity or stage at the design event is increased to undesirable levels, or the opposite bank becomes susceptible to attack. In these cases, the bank upon which the RSP is to be placed must be excavated such that the constructed face of the revetment is flush with the original embankment.

Step 11: Exterior Edges of Revetment. The completed design must be compatible with existing and future conditions. Freeboard and top edge of revetments were covered in Index 873.3(2)(a)(2)(c) "Design Height." For depth of toe, the estimated scour was given as 3.5 feet. This is the minimum toe depth to be considered. Again, based on site conditions and discussions with maintenance staff and others, determine if any long-term conditions need to be addressed. These could include streambed degradation due to local aggregate mining or headcutting. Regardless of the condition, the toe must be founded below the lowest anticipated elevation that could become exposed over the service life of the embankment or roadway facility. As for the upstream and downstream ends, the given length of revetment is 550 feet. Again, this will typically be a minimum, as the designer should seek natural rock outcroppings, areas of quiescent stream flow, or other inherently stable bank segments to end the RSP.

(d) Vegetated Rock Slope Protection. The use of vegetation in streambank stabilization has positive attributes on stream integrity, such as improving stream ecology, increasing soil strength, and providing flow resistance, but vegetation can also have negative impacts on stream integrity by altering conveyance characteristics of the stream, affecting soil characteristics, in addition to being unpredictable in its long term establishment and performance.

Streams with stable vegetation typically have good water quality, as well as good biological and chemical health due in part to the ability of the vegetation to filter pollutants including nitrates and phosphates through their uptake of moisture in the soil. Vegetation will also promote good fish, wildlife, and aquatic organism habitat by providing cover, reducing stream temperature and controlling temperature fluctuations, and supplying an organic food source. In addition to ecological improvements, vegetation can strengthen the underlying soils. It can create additional cohesion and binding properties through its roots. The fibrous woody roots are strong in tension, but weak in compression, which is the opposite case for soil. Therefore, roots and soil working in tandem can complement the other providing a material that has both tension and compression resistance. Vegetation can also improve soil strength by lowering pore-water pressure through its soil moisture extraction.

These benefits of the vegetation root system also carry some negative effects. Their additional mass and surcharge can increase slope failure potential under saturated conditions where the magnitude of saturation can actually be compounded because of root development. Another positive effect of vegetation use in revetments is its ability to improve flow resistance creating higher roughness that will dissipate energy, shear stress, and velocity. The vegetation deflects velocity upwards away from the streambank, which reduces the influence of drag and lift. For example, willows planted on a streambank have the capacity to deflect and resist velocities up to 10 feet/second in their mature state, which would equate to a 12-inch to 18-inch rock (RSP Class III to IV) having similar permissive velocity. To reach this point, it may take three to five years. In the first few years after planting, the vegetation is providing little resistance. During this establishment period, the streambanks can be subject to scour and erosion because of the lack of flow resistance without some other means of protection.

Even after vegetation reaches maturity and beyond, potential exists for it to succumb to drought conditions or to yield to large flows/velocities and break apart rendering the vegetation ineffective to dissipating velocity and hydraulic forces. Because the stages of vegetation growth can be dynamic as it is affected by drought or high flows, the vegetation may go through a reestablishment process, and the n-value and velocity/flow resistance will also be dynamic making revetment performance unpredictable. Even though the use of vegetation in bank stabilization may have negative effects, its ecological benefits generally outweigh them, especially if not located in the vicinity of a bridge, a critical highway structure, or a location where there is a safety concern for highway facilities if the countermeasure fails. If such locations cannot be avoided, a special monitoring and maintenance program may be needed to ensure excessive vegetation growth would not cause adverse effects on the flow characteristics.

The design premise is to use rock and vegetation together in a streambank revetment in such a way that will highlight their positive attributes while also addressing and managing their negative impacts. In the design of hybrid revetments, mounded toes referenced in Index 873.3(3)(a)(1) are not recommended because of their encroachment into the middle of the channel, which can impact cross-sectional area and capacity. With the use of vegetation on the bank and possible projection toward the middle of the channel, cross-sectional area could possibly be impacted as well. A mounded toe used with bank vegetation would only exacerbate this issue, therefore an embedded toe is chosen for hybrid revetment application. See Figure 873.3D for an example cross section of hybrid RSP with an embedded toe. For hybrid revetment design, the 50-year (2% probability) flood event should be used. Per Index 873.2, depending on the importance of the encroachment and level of related risks, subsequent analysis may consider historic high water marks and climate change for design. In order to manage possible negative impacts from vegetation use, planting needs to be performed in a controlled manner. Placement of vegetation within the bank-toe zone and the main channel is highly discouraged to keep turbulence intensity in check that could cause excessive sediment accumulation. Plant mortality must be considered during the initial planting and establishment period. Overplanting must be avoided so that high density and projection does not occur causing increased sediment deposition and capacity/conveyance reduction. Given these issues, plant density in the design of a hybrid revetment and consideration of natural plant density is critical to the performance of the hybrid revetment. The goal for design should be medium density, where horizontal projection and cross-sectional area reduction at maturity are minimal. See Figure 873.3B. For woody vegetation, medium density is described as mature trees or shrubs with full foliage on a streambank, where preferably individual canopies or outer layers retain some free space between them, but may have minimal overlapping without being interwoven.

To provide some protection for vegetation, such as live willow cuttings, and to prevent damage during rock placement, these plants are recommended to be placed inside circular tubes. The tubes would be embedded into the subgrade and allowed to protrude through and extend beyond the RSP outer layer allowing the vegetation within the tube access to adequate sunlight. Although plant tubes provide visual markers for the equipment operator, method A RSP placement type may be necessary to avoid damaging the tubes as rock is placed. Plant tubes should be made of degradable material so that the root and trunk growth are not constrained as the plant matures.

Plant tubes can vary in diameter from 6 to 36 inches. Field fitting the tube lengths may be preferred in order to fit the tubes in its specific project site location. After vegetation and tubes have been placed in the subgrade, the tube can be backfilled with either native soil mixed with water or imported topsoil mixed with water. Over time, the tubes will degrade and will not inhibit the growth and expansion of the vegetation. A variety of plant tubes are on the market; therefore, sole sourcing is not necessary.

The landscape architect should recommend type of plants, and coordinate with the hydraulic engineer to establish tube dimensions and the desired planting pattern and spacing. The hydraulic engineer should analyze plant tubes and the plant's potential full-grown dimensions, considering their effects on the hydraulic capacity and flooding potential to the stream or river. Most vegetation species should not be placed below the normal high-water level. Refer to Design Information Bulletin 87 for Hybrid Streambank Revetment details for Vegetated Rock Slope Protection: https://dot.ca.gov/programs/design/design-information-bulletins-dibs.

Figure 873.3B

Medium Density Vegetation

Lower limit of medium vegetation density

Pre-construction and post- construction hydraulic modelling and hybrid revetment design are discussed in more detail in Design Information Bulletin No. 87. For rock sizing, Index 7.1.1.2 should be substituted with Index 873.3(3)(a)(2)(b) of this manual.

(e) *Gabions*. Gabion revetments consist of rectangular wire mesh baskets filled with stone. See Standard Plan D100A and D100B for gabion basket details and the Standard Specifications for requirements.

Gabions are formed by filling commercially fabricated and preassembled wire baskets with rock. There are two types of gabions, wall type and mattress type. In wall type the empty cells are positioned and filled in place to form walls in a stepped fashion. Mattress type baskets are positioned on the slope and filled. See HEC 23, Volume II, Design Guideline 10 and Figure 873.3B. Wall type revetment is not fully self-adjusting but has some flexibility. The mattress type is very flexible and well suited for man-made roadside channels (with uniform flow) discussed in Chapter 860 and as overside drains that are constructed on steep, unstable slopes. For some stream locations, gabions may be more aesthetically acceptable than rock riprap or may be considered when larger stone sizes are not readily available and flows are nonabrasive. Due to abrasion, corrosion and vandalism concerns and difficulty of repairs, caution is advised regarding in-stream placement of gabions. In addition, the California Department of Fish and Wildlife recommends against using gabions as weirs in streams. If gabions are placed in-stream, some form of abrasion protection in the form of wooden planks or other facing will typically be necessary for wall type, see Figure 873.3C. Maintenance-free design service life in most environments is generally under 20 years.

Figure 873.3C

Gabion Lined Streambank

Gabion wall with timber facing to protect wires from abrasive flow.

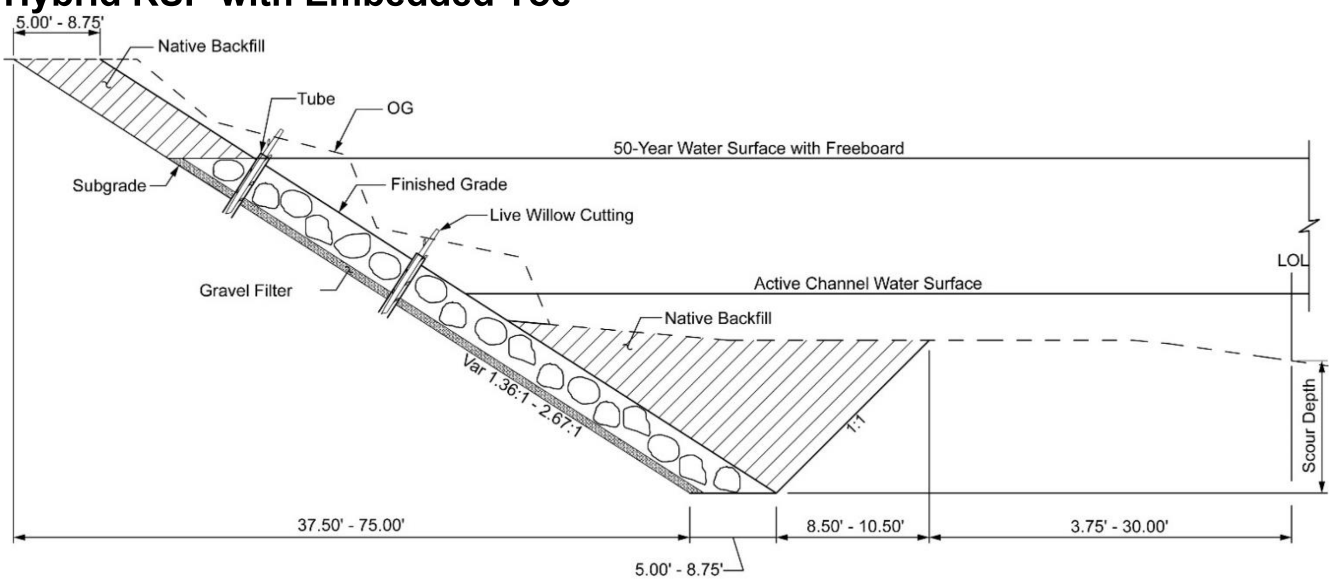
(f) *Articulated Precast Concrete*. This type of revetment consists of pre-cast concrete blocks which interlock with each other, are attached to each other, or butted together to form a continuous blanket or mat. A number of block designs are commercially available. They differ in shape and method of 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

Figure 873.3D

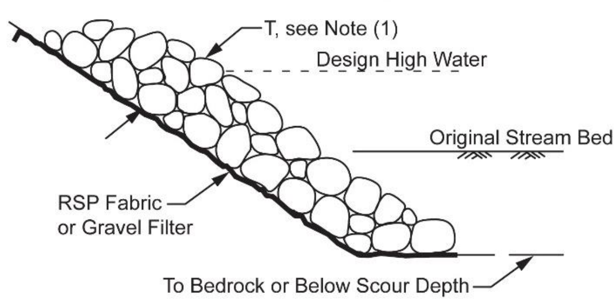
Rock Slope Protection

Hybrid RSP with Embedded Toe

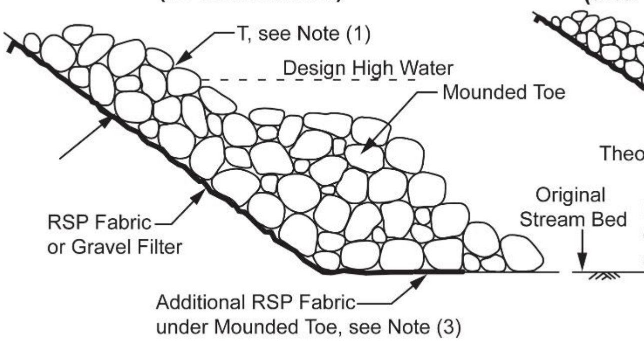


Rock Slope Protection

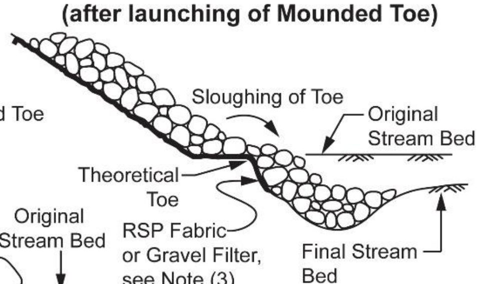
Embedded Toe RSP



Mounded Toe RSP (as constructed)



Mounded Toe RSP (after launching of Mounded Toe)



:

NOTES:
(1)Thickness "T" = 1.5 d50 or d100, whichever is greater.
(2)Face stone size is determined from Index 873.3(2)(a)(2)(a).
(3)RSP fabric not to extend more than 20 percent of the base width of the Mounded Toe past the Theoretical Toe.

generally more aesthetically pleasing than other types of revetment, particularly after vegetation has become established.

Individual blocks are commonly joined together with steel cable or synthetic rope, to form articulated block mattresses. Pre-assembled in sections to fit the site, the mattresses can be used on slopes up to 2:1. They are anchored at the top of the revetment to secure the system against slippage.

Pre-cast block revetments that are formed by butting individual blocks end to end, with no physical connection, should not be used on slopes steeper than 3:1. An engineering fabric is normally used on the slope to prevent the migration 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.

(4) Rigid Revetments.

(a) Concreted-Rock Slope Protection.

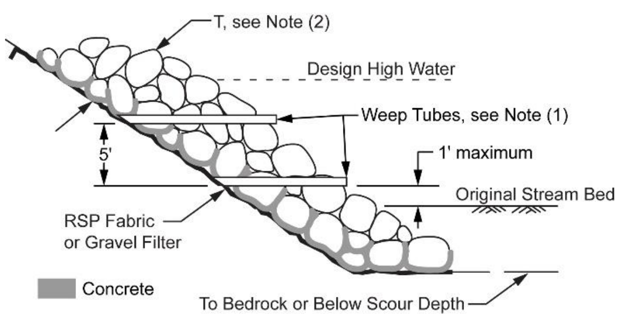
(1) General Features. This type of revetment consists of rock slope protection with interior voids filled with PCC to form a monolithic armor. A typical section of this type of installation is shown in Figure 873.3E.

It has application in areas where rock of sufficient size for ordinary rock slope protection is not economically available.

(2) Design Concepts. Concreting of RSP is a common practice where availability of large stones is limited, or where there is a need to reduce the total thickness of a RSP revetment. Inclusion of the concrete, and the labor required to place it, makes concreted RSP installations more expensive per unit area than non-concreted installations.

Figure 873.3E

Concreted-Rock Slope Protection



NOTES:

(1) If needed to relieve hydrostatic pressure.

(2) $1.5d_{50}$ or d_{100}, whichever is greater from Table 873.3A for section thickness.

Dimensions and details should be modified as required.

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(3)(a)(2)(c) to select a stable rock class for a non-concreted design based on the d_{50} and the next larger class in Table 873.3A. This non-concreted rock size is divided by a factor of roughly four or five to arrive at the appropriate d_{50} size 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 d_{50} rock to select the appropriate RSP Class.

As this type of protection is rigid without high strength, support by the embankment must be maintained. Slopes steeper than the angle of repose of the embankment are risky, but with rocks grouted in place, little is to be gained with slopes flatter than 1.5:1. Precautions to prevent undermining of embankment are particularly important, see Figure 873.3F. The concreted-rock must be founded on solid rock or below the depth of possible scour. Ends should be protected by tying into stable rock or forming smooth transitions with embankment subjected to lower velocities. As a precaution, cutoff stubs may be provided. If the embankment material is exposed at the top, freeboard is warranted to prevent overtopping.

Figure 873.3F

Toe Failure – Concreted RSP

Toe of concreted RSP that has been undermined.

The design intent is to place an adequate volume of concrete to tie the rock mass together, but leave the outer face roughened with enough rock projecting above the concrete to slow flow velocities to more closely approximate natural conditions.

The volume of concrete required is based on filling roughly two-thirds of the void space of the rock layer, as shown in Figure 873.3E. The concrete is rodded or vibrated into place leaving the outer stones partially exposed. Void space for the various RSP gradations ranges from approximately 30 percent to 35 percent for Method A placed rock to 40 percent to 45 percent for Method B placed rock of the total volume placed.

Specifications. Quality specifications for rock used in concreted-rock slope protection are usually the same as for rock used in ordinary rock slope protection. However, as the rocks are protected by the concrete which surrounds them, specifications for specific gravity and hardness may be lowered if necessary. The concrete used to fill the voids is normally 1 inch maximum size aggregate minor concrete. Except for freeze-thaw testing of aggregates, which may be waived in the contract special provisions, the concrete should conform to the provisions of Standard Specification Section 90.

Size and grading of stone and concrete penetration depth are provided in Standard Specification Section 72.

(b) Partially Grouted Rock Slope Protection. Partially grouted rock slope protection (PGRSP) is a viable alternative to larger rock or concreted rock slope protection where either the availability of large material is limited, or site limitations regarding placement of large material (e.g., no excavation below spread footing base) would lead the designer to consider using some form of smaller rock held together with a cementitious material. With partially grouted rock slope protection, there are no relationships per se for selecting the size of rock, other than the practical considerations of proper void size, gradation, and adequate stone-to-stone contact area. The intent of partial grouting is to "glue" stones together to create a conglomerate of particles. Each conglomerate is therefore significantly larger than the d_{50} stone size, and typically is larger than the d_{100} size of the individual stones in the matrix. The proposed gradation criteria are based on a nominal or "target" d_{50} and only stones with a d_{50} ranging from 9 inches to 15 inches may be used with the partial grouting technique. See rock classes II, III and IV in Table 873.3A. In HEC 23, PGRSP is presented as a pier scour countermeasure, but it may be also used for bridge abutment protection, as well as for bed/bank protection for short localized areas with high velocities and shear stresses that require a smaller rock

footprint than a non-grouted design. Both Headquarters Office of Highway Drainage Design and District biologist staff should be consulted early on during the planning phase for subject matter expertise relative to design and obtaining project specific permits. For more guidance, see HEC 23, Volume 2, Design Guideline 12.

(c) Sacked-Concrete Slope Protection. This method of protection consists of facing the embankment with sacks filled with concrete. It is expensive, but historically was a much used type of revetment. Much hand labor is required but it is simple to construct and adaptable to almost any embankment contour. Use of this method of slope protection is generally limited to replacement or repair of existing sacked concrete facilities, or for small, unique situations that lend themselves to hand-placed materials.

Tensile strength is low and as there is no flexibility, the installation must depend almost entirely upon the stability of the embankment for support and therefore should not be placed on face slopes much steeper than the angle of repose of the embankment material. Slopes steeper than 1:1 are rare; 1.5:1 is common. The flatter the slope, the less is the area of bond between sacks. From a construction standpoint it is not practical to increase the area of bond between sacks; therefore for slopes as flat as 2:1 all sacks should be laid as headers rather than stretchers.

Integrity of the revetment can be increased by embedding dowels in adjoining sacks to reinforce intersack bond. A No. 3 deformed bar driven through a top sack into the underlying sack while the concrete is still fresh is effective. At cold joints, the first course of sacks should be impaled on projecting bars that were driven into the last previously placed course. The extra strength may only be needed at the perimeter of the revetment.

Most failures of sacked concrete are a result of stream water eroding the embankment material from the bottom, the ends, or the top.

The bottom should be founded on bedrock or below the depth of possible scour.

If the ends are not tied into rock or other nonerosive material, cutoff returns are to be provided and if the protection is long, cutoff stubs are built at 30-foot intervals, in order to prevent or retard a progressive failure.

Protection should be high enough to preclude overtopping. If the roadway grade is subject to flooding and the shoulder material does not contain sufficient rock to prevent erosion from the top, then pavement should be carried over the top of the slope protection in order to prevent water entering from this direction.

Class 8 RSP fabric as described in Standard Specification Section 96 should be placed behind all sacked concrete revetments. For revetments over 4 feet in height, weep tubes should also be placed, see Figure 873.3E.

For good appearance, it is essential that the sacks be placed in horizontal courses. If the foundation is irregular, corrective work such as placement of entrenched concrete or sacked concrete is necessary to level up the foundation. Refer to HDS No. 6, Section 6.6.5, for further discussion on the use of sacked concrete slope protection.

(5) *Bulkheads.* A bulkhead is a steep or vertical structure supporting a natural slope or constructed embankment. As bank protection structures, bulkheads serve to secure the bank against erosion as well as retaining it against sliding. As a retaining structure, conventional design methods for retaining walls, and laterally loaded piles are used.

Bulkheads are usually expensive, but may be economically justified in special cases where valuable riparian property or improvements are involved and foundation conditions are not satisfactory for less expensive types of slope protection. They may be used for toe protection in combination with other revetment types of slope protection. Some other considerations that may justify the use of bulkheads include:

- Encroachment on a channel cannot be tolerated.
- Retreat of highway alignment is not viable.
- Right of Way is restricted.
- The force and direction of the stream can best be redirected by a vertical structure.

The foundation for bulkheads must be positive and all terminals secure against erosive forces. The length of the structure should be the minimum necessary, with transitions to other less expensive types of slope protection when possible. Eddy currents can be extremely damaging at the terminals and transitions. If overtopping of the bulkheads is anticipated, suitable protection should be provided.

Along a stream bank, using a bulkhead presumes a channel section so constricted as to prohibit use of a cheaper device on a natural slope. Velocity will be unnaturally high along the face of the bulkhead, which must have a fairly smooth surface to avoid compounding the restriction. The high velocity will increase the threat of scour at the toe and erosion at the downstream end. Allowance must be made for these threats in selecting the type of foundation, grade of footing, penetration of piling, transition, and anchorage at downstream end. Transitions at both ends may appropriately taper the width of channel and slope of the bank. Transition in roughness is desirable if attainable. Refer to HDS No. 6, Section 6.4.8, for further discussion on the use of bulkheads to prevent streambank erosion or failure.

(a) Concrete or Masonry Walls. The expertise and coordination of several engineering disciplines is required to accomplish the development of PS&E for concrete walls serving the dual purpose of slope protection and support. The Division of Structures is responsible for the structural integrity of all retaining walls, including bulkheads.

(b) Sheet Piling. Timber, concrete and steel sheet piling are used for bulkheads that depend on deep penetration of foundation materials for all or part of their stability. High bulkheads are usually counterforted at upper levels with batter piles or tie back systems to deadmen. Any of the three materials is adaptable to sheet piling or a sheathed system of post or column piles.

Excluding structural requirements, design of pile bulkheads is essentially as follows:

- Recognition of foundation conditions suitable to or demanding deep penetration. Penetration of at least 15 feet below scour level, or into soft rock, should be assured.
- Choice of material. Timber is suitable for very dry or very wet climates, for other situations economic comparison of preliminary designs and alternative materials should be made.
- Determination of line and grade. Fairly smooth transitions with protection to high-water level should be provided.

(6) Vegetation. Vegetation is the most natural method for stabilization of embankments and channel bank protection. Vegetation can be relatively easy to maintain, visually attractive and environmentally desirable. The root system forms a binding network that helps hold the soil. Grass and woody plants above ground provide resistance to the near bank water flow causing it to lose some of its erosive energy.

Erosion control and revegetation mats are flexible three-dimensional mats or nets of natural or synthetic material that protect soil and seeds against water erosion prior to establishment of vegetation. They permit vegetation growth through the web of the mat material and have been used as temporary channel linings where ordinary seeding and mulching techniques will not withstand erosive flow velocities. The designer should recognize that flow velocity estimates and a particular soils resistance to erosion are parameters that must be based on specific site conditions. Using arbitrarily selected values for design of vegetative slope protection without consultation with the District Hydraulic Unit and/or the District Landscape Architect Unit is not recommended. However, a suggested starting point of reference is Table 865.2 in which the resistance of various unprotected soil classifications to flow velocities are given. Under near ideal conditions, ordinary seeding and mulching methods cannot reasonably be expected to withstand sustained flow velocities above 4 feet per second. If velocities are in excess of 4 feet per second, a lining maybe needed, see Table 865.2.

Temporary channel liners are used to establish vegetative growth in a drainage way or as slope protection prior to the placement of a permanent armoring. Some typical temporary channel liners presented in Table 865.2 are:

- Single net straw
- Double net coconut/straw blend
- Double net shredded wood

Vegetative and temporary channel liners are suitable for conditions of uniform flow and moderate shear stresses.

Permanent soil reinforcing mats and rock riprap may serve the dual purpose of temporary and permanent channel liner. Some typical permanent channel liners are:

- Small rock slope protection
- Geosynthetic mats
- Polyethelene cells or grids
- Gabion Mattresses (see Index 873.3(3)(a)(2)(e))

However, geosynthetics and plastic (polyethylene, polypropylene, polyamide, etc.) based mats with no enhanced UV resistance must be installed in a fashion where there will be no potential for long-term sunlight exposure, as these products will degrade due to UV radiation.

Composite designs are often used where there are sustained low flows of high to moderate velocities and intermediate high water flows of low to moderate velocities. Brush layering is a permanent type of erosion control technique that may also have application for channel protection, particularly as a composite design.

Additional design information on vegetation, and temporary and permanent channel liners is given in Chapter IV, HEC 15, Design of Roadside Channels and Flexible Linings and in Chapter 860 of this manual.

873.4 Training Systems

(1) *General*. Training systems are structures, usually within a channel, that act as countermeasures to control the direction, velocity, or depth of flowing water. When training systems are used, they generally straighten the channel, shorten the flow line, and increase the local velocity within the channel. Any such changes made in the system that cause an increase in the gradient may cause an increase in local velocities. The increase in velocity increases local and contraction scour with subsequent deposition downstream, where the channel takes on its normal characteristics. If significant lengths of the river are trained and straightened, there can be a noticeable decrease in the elevation of the water surface profile for a given discharge in the main channel. Tributaries emptying into the main channel in such reaches are significantly affected. Having a lower water level in the main channel for a given discharge means that the tributary streams entering in that vicinity are subjected to a steeper gradient and higher velocities which can cause degradation in the tributary streams. In extreme cases, degradation can be induced of such magnitude as to cause failure of structures such as bridges, culverts or other encroachments on the tributary systems. In general, any increase in transported materials from the tributaries to the main channel causes a reduction in the quality of the environment within the river.

(a) Bendway Weirs. Bendway weirs, also referred to as stream barbs, bank barbs, and reverse sills are low elevation stone sills used to improve lateral stream stability and flow alignment problems at river bends and highway crossings on streams and smaller rivers.

They are placed at an angle with the embankment in meandering streams for the purpose of directing or forcing the current away from the embankment, see Figure 873.4A. They also encourage deposition of bed material and growth of vegetation. When the purpose is to deposit material and promote growth, the weirs are considered to have fulfilled their function and are expendable when this occurs.

Figure 873.4A

Thalweg Redirection Using Bendway Weirs

Bendway weirs in conjunction with rock slope protection.

Bendway weirs are similar in appearance to stone spurs, but have significant functional differences. Spurs are typically visible above the flow line and are designed so that flow is either diverted around the structure, or flow along the bank line is reduced as it passes through the structure. Bendway weirs are normally not visible, especially at stages above low water, and are intended to redirect flow by utilizing weir hydraulics over the structure. Flow passing over the bendway weir is redirected such that it flows perpendicular to the axis of the weir and is directed towards the channel centerline. See Figure 873.4B for typical cross section and layout. Similar to stone spurs, bendway weirs reduce near bank velocities, reduce the concentration of currents on the outer bank, and can produce a better alignment of flow through the bend and downstream crossing. Experience with bendway weirs has indicated that the structures do not perform well in degrading or sediment deficient reaches.

Material sizing should be based on the Isbash equation plotted in Figure 873.4C. Riprap stone size is designed using the critical velocity near the boundary where the riprap is placed. Typically the size ranges between 1 and 3 ft and should be approximately 20% greater than that computed from the rock sizing formula presented in Index 873.3(3)(a)(2)(b). The minimum rock size should not be less than the D_{100} of the streambed material. See Tables 873.3A and 873.3B to determine rock class.

See HEC 23 Volume 2, Design Guideline 1 for detailed guidance on weir height, length, angle, location and spacing,

(b) Spurs. A spur can be a pervious or impervious structure projecting from the streambank into the channel. Similar to bendway weirs, spurs are used to halt meander migration at a bend and channelize wide, poorly defined streams into well-defined channels by reducing flow velocities in critical zones near the streambank to prevent erosion and establish a more desirable channel alignment or width. The main function of spurs is to reduce flow velocities near the bank, which in turn, encourages sediment deposition due to these reduced velocities. Increased protection of banks can be achieved over time, as more sediment is deposited behind the spurs. Because of this, spurs may protect a streambank more effectively and at less cost than revetments. Furthermore, by moving the location of any scour away from the bank, partial failure of the spur can often be repaired before damage is done to structures along and across the stream.

In braided streams, the use of spurs to establish and maintain a well-defined channel location, cross section, and alignment can decrease the required bridge length, thus decreasing the cost of bridge construction and maintenance.

Spur types are classified based upon their permeability as retarder spurs, retarder/deflector spurs, and deflector spurs. The permeability of spurs is defined simply as the percentage of the spur surface area facing the streamflow that is open. Deflector spurs are impermeable spurs which function by diverting the primary flow currents away from the bank. Retarder/deflector spurs are more permeable and function by retarding flow velocities at the bank and diverting flow away from the bank. Retarder spurs are highly permeable and function by retarding flow velocities near the bank.

These structures should be designed not to overtop. Therefore, for permeable spurs, the rock sizing formula presented in Index 873.3(3)(a)(2)(b) may be used and a Cv value of 1.25 is recommended. Where overtopping the spur is unavoidable, the riprap size may be determined by equations 5.2 (for slopes > 25%) or 5.3 (for slopes < 25%) in HEC 23 Volume 2, Design Guideline 5. Since these equations are for free flow down the slope, always check to see if the structure is actually drowned (submerged) by high tailwater. If that is the case, then use the rock sizing formula presented in Index 873.3(3)(a)(2)(b) for sizing riprap on a stream bank should be used. See Tables 873.3A and 873.3B to determine rock class.

In general a top width equal to the width of a dump truck can be used. The side slopes of the spur should be 2H:1V or flatter. Rock riprap should be placed on the upstream and downstream faces as well as on the nose of the spur to inhibit erosion of the spur.

Figure 873.4B

Bendway Weir Typical Cross Section and Layout

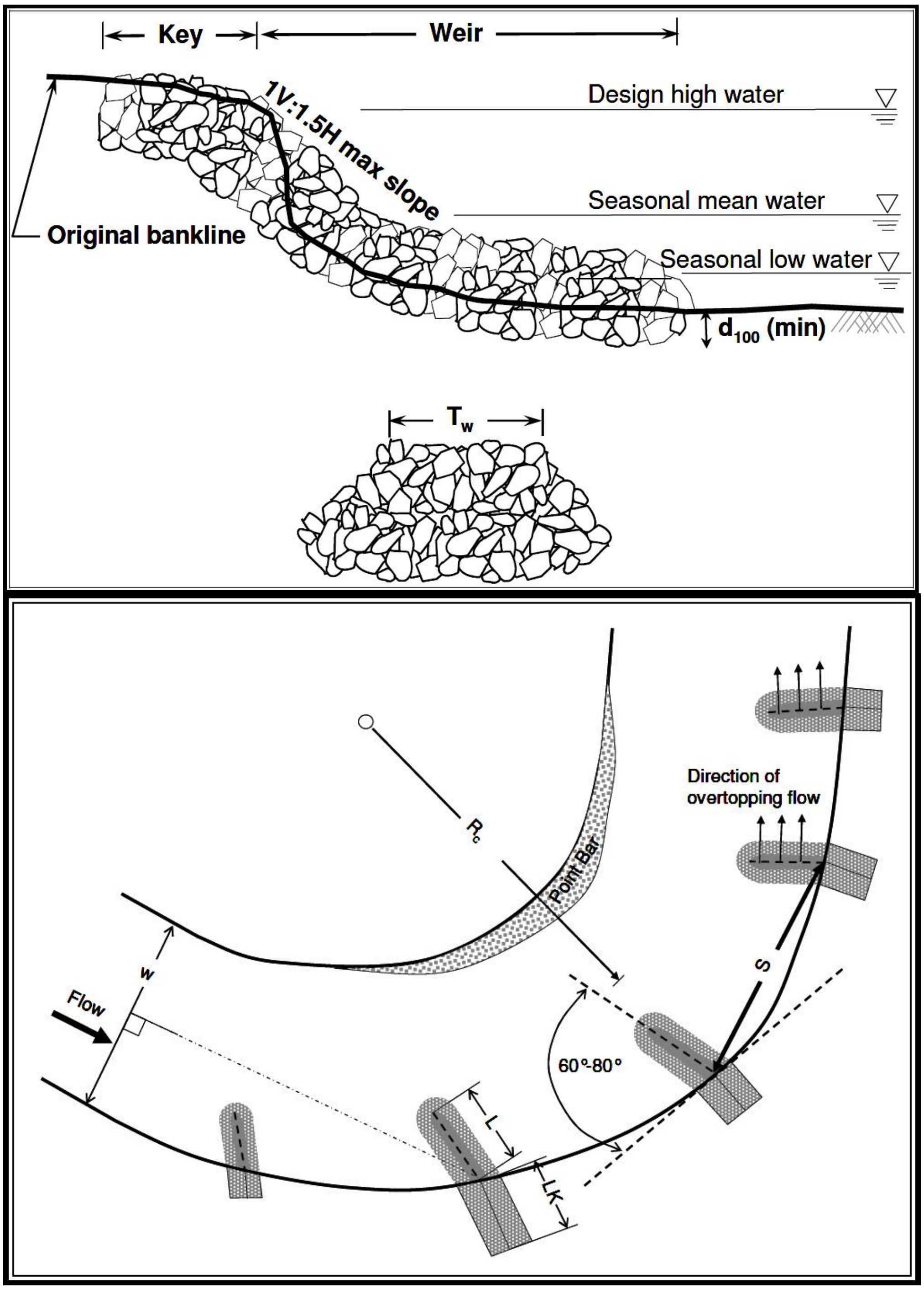
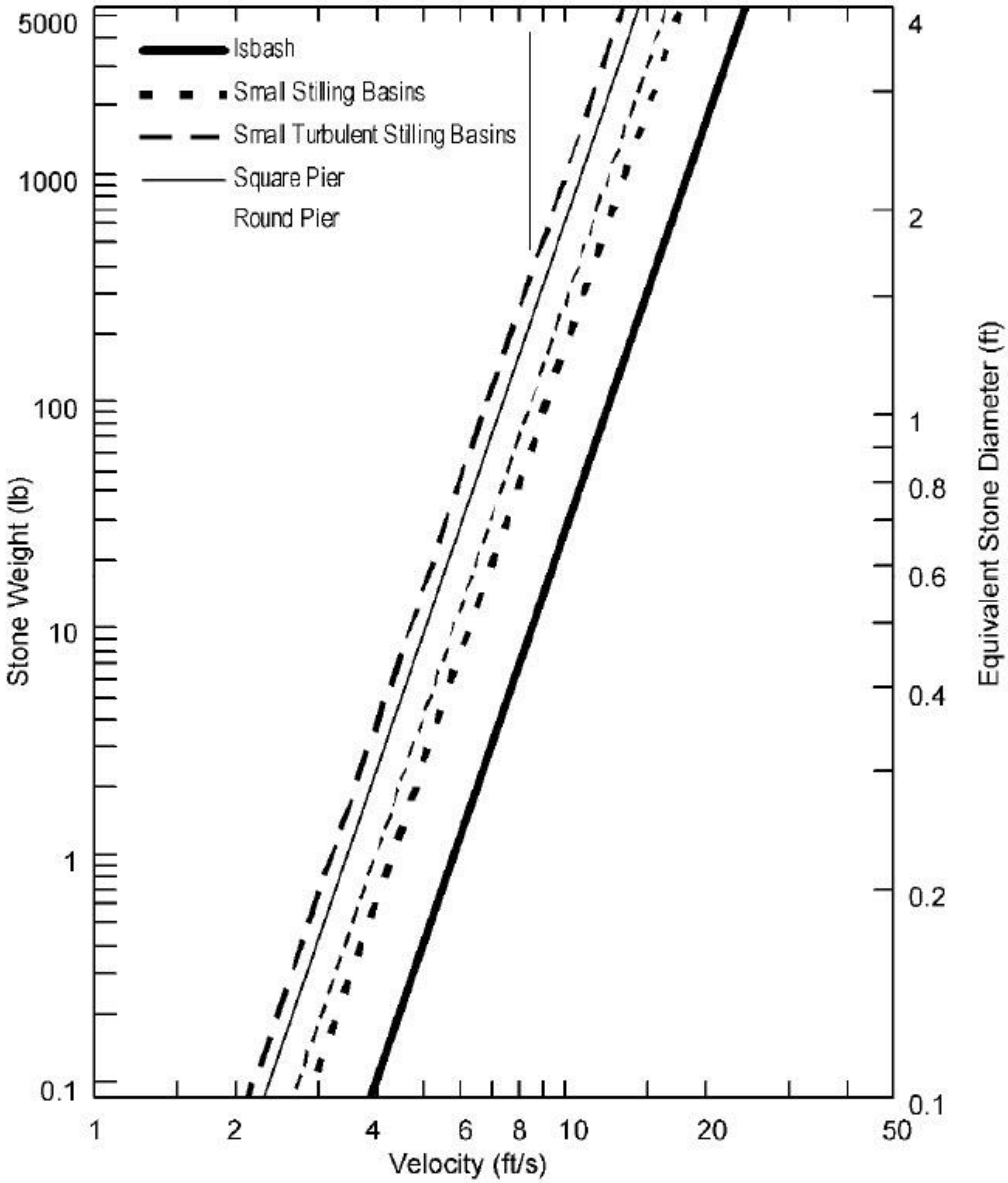


Figure 873.4C

Bendway Weir Rock Size Chart

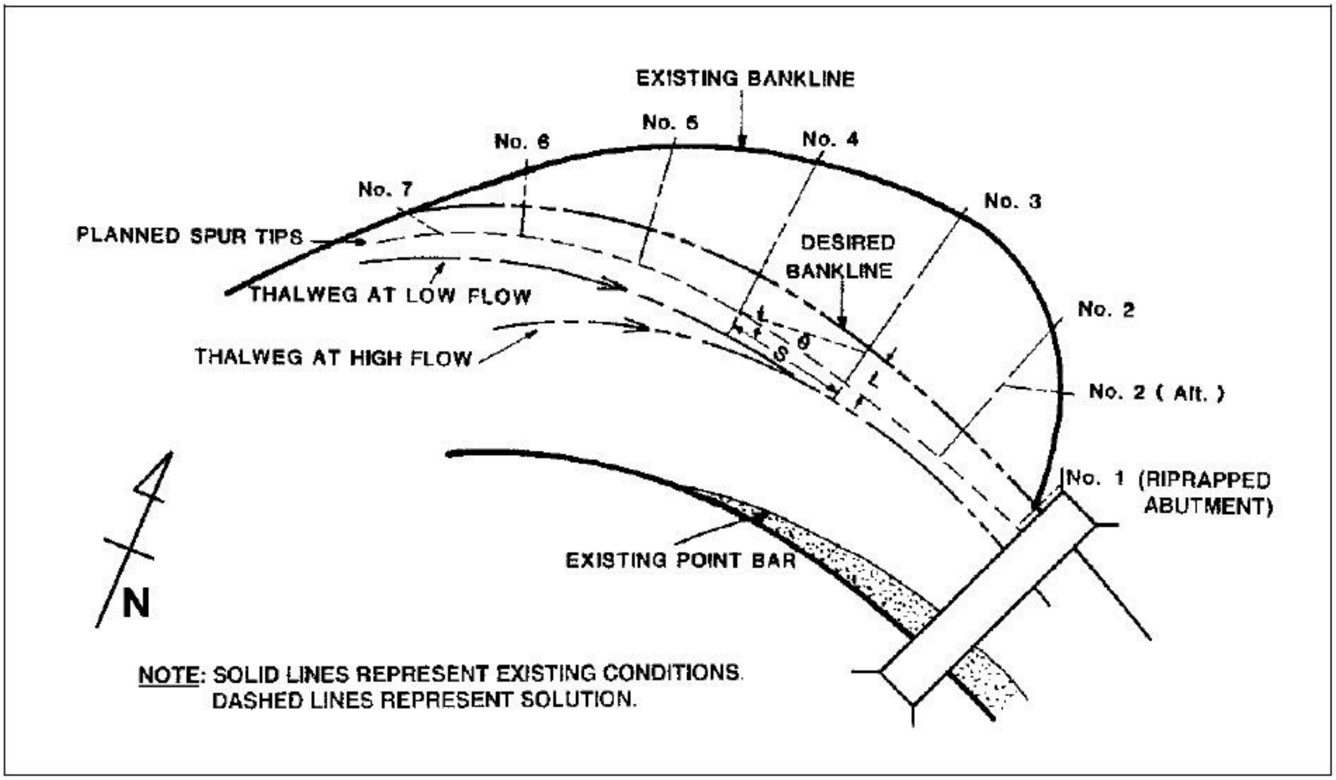


Depending on the embankment material being used, a gravel, sand, or geotextile filter may be required. It is recommended that riprap be extended below the bed elevation to the combined long-term degradation and contraction scour depth. Riprap should also extend to the crest of the spur, in cases where the spur would be submerged at design flow, or to 2 feet above the design flow, if the spur crest is higher than the design flow depth. Additional riprap should be placed around the nose of the spur, so that spur will be protected from scour.

See Figure 873.4D for example of spur design and HEC 23 Volume 2, Design Guideline 2, for detailed guidance on spur height, length, shape, angle, permeability, location and spacing.

Figure 873.4D

Example of Spur Design



(c) Guide Dikes/Banks. Guide banks are appendages to the highway embankment at bridge abutments, see Figure 873.4E. They are smooth extensions of the fill slope on the upstream side. When embankments encroach on wide floodplains to attain an economic length of bridge, the flows from these areas must flow parallel to the approach embankment to the bridge opening. These flows can cause a severe flow contraction at the abutment with damaging eddy currents that can scour away abutment and pier foundations, erode the approach embankment, and reduce the effective bridge opening.

Guide banks can be used in these cases to prevent erosion of the approach embankments by cutting off the flow adjacent to the embankment, guiding streamflow through a bridge opening, and transferring scour away from abutments to prevent damage caused by abutment scour. The two major enhancements guide banks bring to bridge design are (1) reduce the separation of flow at the upstream abutment face and thereby maximize the use of the total bridge waterway area, and (2) reduce the abutment scour due to lessening turbulence at the abutment face. Guide banks can be used on both sand and gravel-bed streams.

Guide banks are usually earthen embankment faced with rock slope protection. Optimum shape and length of guide dikes will be different for each site. Field experience has shown that an elliptical shape with a major to minor axis ratio of 2.5:1 is effective in reducing turbulence. The length is dependent on the ratio of flow diverted from the floodplain to flow in the first 100 feet of waterway under the bridge. If the use of another shape dike, such as a straight dike, is required for practical reasons more scour should be expected at the upstream end of the dike. The bridge end will generally not be immediately threatened should a failure occur at the upstream end of a guide dike.

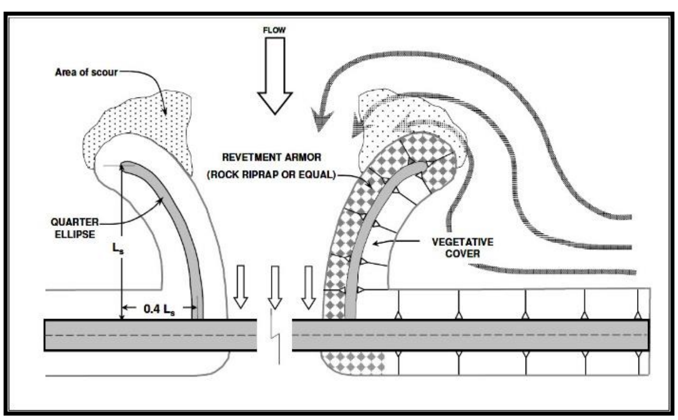
Toe dikes are sometimes needed downstream of the bridge end to guide flow away from the structure so that redistribution in the floodplain will not cause erosion damage to the embankment due to eddy currents. The shape of toe dikes is of less importance than it is with upstream guide banks.

Principal factors to be considered when designing guide banks, are their orientation to the bridge opening, plan shape, upstream and downstream length, cross-sectional shape, and crest elevation.

It is apparent from the Figure 873.4E that without this guide bank, overbank flows would return to the channel at the bridge opening, which can increase the severity of contraction and scour at the abutment. With installation of guide banks the scour holes which normally would occur at the abutments of the bridge are moved upstream away from the abutments. Guide banks may be designed at each abutment, as shown, or singly, depending on the amount of overbank or floodplain flow directed to the bridge by each approach embankment.

Figure 873.4E

Bridge Abutment Guide Banks



The goal in the design of guide banks is to provide a smooth transition and contraction of the streamflow through the bridge opening. Ideally, the flow lines through the bridge opening should be straight and parallel. As in the case with other countermeasures, the designer should consider the principles of river hydraulics and morphology, and exercise sound engineering judgment.

The Division of Engineering Services (DES) and Structures Maintenance and Investigations (SMI) Hydraulics Branches are responsible for the hydraulic design of bridges, therefore, for protection at bridge abutments and approaches, the District is responsible for consulting with them to verify the design parameters and also obtaining the bridge hydraulic model. See Index 873.6 “Coordination with the Division of Engineering Services and Structures Maintenance and Investigations.”

For further detailed information on guide bank design procedures, refer to HEC 23, Volume 2, Design Guidelines 14 and 15. See Tables 873.3A and 873.3B to determine rock class.

(d) Further Information and Other Countermeasures for Lateral Stream Instability. General design considerations and guidance for evaluating scour and stream stability at highway bridges is contained in HEC 18, HEC 20, and HEC 23.

For further information on other countermeasures such as retarder structures, longitudinal dikes and bulkheads, see HEC 23 Volume 1, Chapter 8.

(e) Check Dams and Drop Structures. Drop structures or check dams are an effective means of gradient control. They may be constructed of rock, gabions, concrete, treated timber, sheet piling or combinations of any of the above. They are most suited to locations where bed materials are relatively impervious otherwise underflow must be prevented by cutoffs. Rock riprap and timber pile construction have been most successful on channels having small drops and widths less than 100 ft. Sheet piles, gabions, and concrete structures are generally used for larger drops on channels with widths ranging up to 300 ft. Check dams can initiate erosion of banks and the channel bed downstream of the structure as a result of energy dissipation and turbulence at the drop. This local scour can undermine the check dam and cause failure. The use of energy dissipators downstream of check dams can reduce the energy available to erode the channel bed and banks. In some cases it may be better to construct several consecutive drops of shorter height to minimize erosion. Lateral erosion of channel banks just downstream of drop structures is another adverse result of check dams and is caused by turbulence produced by energy dissipation at the drop, bank slumping from local channel bed erosion, or eddy action at the banks. The usual solution to these problems is to place rock slope protection on the streambank adjacent to the drop structure or check dam. Erosion of the streambed can also be reduced by placing rock riprap in a preformed scour hole downstream of the drop structure. A row of sheet piling with top set at or below streambed elevation can keep the riprap from moving downstream. Because of the problems associated with check dams, the design of these countermeasures requires designing the check dams to resist scour by providing for dissipation of excess energy and protection of areas of the bed and the bank which are susceptible to erosive forces. Refer to HEC 23 Volume 2, Design Guideline 3 and HDS No. 6, Section 6.4.11, for further discussion on the use of check dams and drop structures.

873.5 Summary and Design Check List

The designer should anticipate the more significant problems that are likely to occur during the construction and maintenance of channel protection facilities. So far as possible, the design should be adjusted to eliminate or minimize those potential problems.

The logistics of the construction activity such as access to the site, on-site storage of construction materials, time of year restrictions, environmental concerns, project specific permits and sequence of construction should be carefully considered during the project design.

See Index 872.1, Planning, Index 872.3(6), Construction, Easements, Access and Staging, and Index 872.3(7), Biodiversity. The stream morphology and its response to construction activities is an integral part of the planning process. Communication between the designer and those responsible for construction administration as well as maintenance are important.

Channel protection facilities require periodic maintenance inspection and repair. Where practicable, provisions should be made in the facility design to provide access for inspection and maintenance.

The following check list has been prepared for both the designer and reviewer. It will help assure that all necessary information is included in the plans and specifications. It is a comprehensive list for all types of protection. Items pertinent to any particular type can be selected readily and the rest ignored.

(1) Location and staging of the planned work with respect to:

- The highway.
- The stream, its morphology, biodiversity and project specific permits.
- Right of way. See Index 872.1 and 872.3 for construction easements and examination of stream behavior far upstream and downstream.

(2) Datum control of the work, and relation of that datum to gage datum on streams.

(3) A typical cross section indicating dimensions, slopes, arrangement and connections.

(4) Quantity of materials (per foot, per protection unit, or per job).

(5) Relation of the foundation treatment with respect to the existing ground.

(6) Relation of the top of the proposed protection to design high water (historic, with date; or predicted, with frequency).

(7) The limits of excavation and backfill as they may affect measurement and payment.

(8) Construction details such as weep holes, rock slope protection fabrics, geocomposite drains and associated materials.

(9) Location and details of construction joints, cut-off stubs and end returns.

(10) Restrictions to the placement of reinforcement.

(11) Connections and bracing for framing of timber or steel.

(12) Splicing details for timber, pipe, rails and structural shapes.

(13) Anchorage details, particularly size, type, location, and method of connection.

(14) Size, shape, and special requirements of units such as precast concrete shapes and other manufactured items.

(15) Number and arrangement of cables and details of fastening devices.

(16) Size, mass per unit area, mesh spacing and fastening details for wire-fabric or geosynthetic materials.

(17) On timber pile construction the number of piles per bent, number of bents, length of piling, driving requirements, cut-off elevations, and framing details.

(18) The details of gabions and the filling material. See Standard Plan D100A and D100B and the Standard Specifications.

*(19)*The size of articulated blocks, the placement of steel, and construction details relating to fabrication.

*(20)*The corrosion considerations that may dictate specialty concretes, coated reinforcing, or other special requirements.

873.6 Coordination with the Division of Engineering Services and Structures Maintenance and Investigations

(1) The Division of Engineering Services and Structures Maintenance and Investigations Hydraulics Branches. The Division of Engineering Services (DES) and Structures Maintenance and Investigations (SMI) Hydraulics Branches are responsible for the hydraulic design of bridges. Therefore, for protection at bridge piers, abutments and approaches, the District is responsible for consulting with them to verify the design parameters (i.e., water surface elevations, freeboard requirements, water velocities, scour recommendations etc.) used and also obtaining the bridge hydraulic model.

Figure 873.6A

Bridge Abutment Failure Example

Bridge Abutment Failure at Tex Wash on I-10 after a Flood Larger Than the Design Flood

The DES Hydraulics branch performs all hydraulic designs for new bridges or replacement bridges that meet the National Bridge Inventory (NBI) bridge definition. Modifications to an existing bridge or constructing a new bridge require obtaining permits from the regulatory agencies. The DES Hydraulics branch should coordinate with the District to perform conceptual designs for permit approval. The DES Hydraulics branch is essentially a consultant/designer to the District Design Offices.

The SMI Hydraulics branch within the Division of Maintenance is responsible for the hydraulic analyses, repair and monitoring of in-service bridges. Typical maintenance challenges include scour, flooding, and lateral migration. Maintenance related impacts to a bridge will trigger a hydraulic report for that specific bridge. The hydraulic report recommendations are used by the District in determining the scope of hydraulic improvements to the bridge projects. For countermeasure design at bridge abutments and piers (e.g., rock slope protection, guide banks, check dams, structural repairs etc.) the magnitude of the discharge used is the 100-year flood. This standard is independent of the design flood used by the District for protecting the channel bank or the bridge approach embankment (see Index 873.2).

Since the mid 1990's, new bridges have been designed so that the top of the pile cap is at the bottom of anticipated scour (long-term degradation, contraction and local scour) for the 100-year flood using the hypothesis that by designing the foundations lower than recommended in HEC 18 for the 100-year flood, there would be ample safety factor inherent to withstand the 200-year scour check flood. Bridges that were designed prior to the first edition of HEC 18 in 1991 may be more vulnerable to the possible effects of climate change or floods larger than the 100-year flood. See Figure 873.6A.

Depending on location, site considerations may include constructability and biodiversity, see Index 872.3(4) and Index 872.3(5). During the planning and environmental phases on environmentally sensitive projects (e.g., bridge structures that require permits for fish passage design under California Fish and Wildlife jurisdiction – see Figure 873.3B), the District should initiate contact with resource agencies early to propose conceptual design, identify impacts and any necessary mitigation as part of the permitting process. The overriding issue of concern is the difference in timing of detailed analyses (e.g., hydraulics, geotechnical, foundation) that takes place on the District side of the project development process verses what takes place in DES prior to project approval during the environmental phase.

Prematurely approved projects and environmental documents prior to permit approval can, and do, result in costly major re-work during the design phase. On environmentally sensitive projects the District should consider the need to shift resources to the environmental phase so that a more advanced bridge foundation design can be incorporated into the Advanced Planning Study (APS) and the Environmental Document (ED) to facilitate permit approval consistent with the Project Approval (PA) and to minimize rework.

Figure 873.6B

Habitat Enhancement Example

Longitudinal Peaked Stone Toe Protection (LPSTP) with Rock Vanes for Chinook Salmon Habitat Enhancement, Route 128, Russian River Bridge in Geyserville

(2) *Geotechnical Design and Geology.* The Project Engineer must review the Project Initiation Document and Preliminary Geotechnical Design Report, if any, to ascertain the scope of geotechnical involvement for a project.

For all projects that involve designs for cut slopes, embankments, earthwork, landslide remediation, retaining walls, groundwater studies, erosion control features, subexcavation and any other studies involving geotechnical investigations and engineering geology, a Geotechnical Design Report (GDR) is to be prepared by the Roadway Geotechnical Engineering Branches of the Division of Engineering Services, Geotechnical Services (DES-GS).

Coordination with Geotechnical Design and Geology within DES may be initiated by the designer when any of the following determinations need to be made:

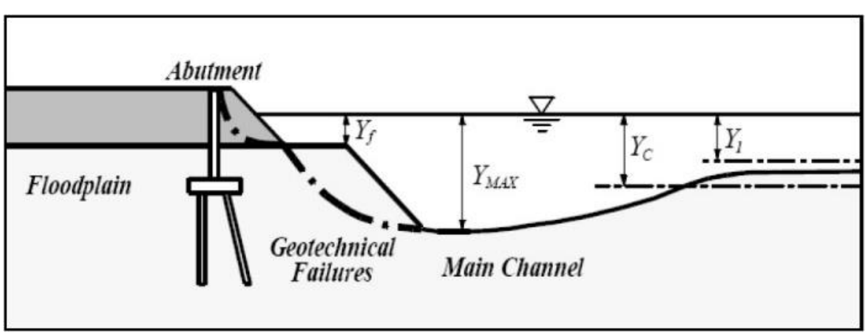
- Scour potential of channel material.
- Natural erosion potential of stream banks that may affect project features. See Figure 873.6C.
- The performance of existing cut, fill and natural slopes including the slope soil/rock composition.
- Slope stability analysis and need for earth retaining systems including gabion walls.
- Embankment constructability and impact to nearby structures or bridge abutments. See following link to the Geotechnical Manual and Figure 873.6D: https://dot.ca.gov/programs/engineering-services/manuals/geotechnical-manual.

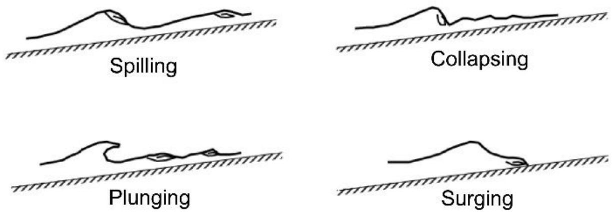
Figure 873.6C

Lateral Stream Migration Within a Canyon Setting Example

Figure 873.6D

Conceptual Geotechnical Failures Resulting from Abutment Scour





The Pilarczyk method, like the Hudson method, uses a general empirical relationship for particle stability under wave action. When design wave heights are much greater than H=5 feet, contact the District Hydraulic Engineer. The Pilarczyk equation is:

$$\frac{H_s}{\Delta D} \leq \psi_u \phi \frac{\cos\theta}{\xi^b}$$

Where:

H_s = significant wave height, (ft)

Δ = relative unit weight of riprap, $\Delta = (\gamma_r - \gamma_w)/\gamma_w$

D = armor size thickness, (ft)

ψ_u = stability upgrade factor (1.0 for good riprap)

ϕ = stability factor (1.5 for good quality, angular riprap)

θ = angle of slope inclination

ξ = dimensionless breaker parameter

b = exponent (0.5 for riprap)

Rearranging the Pilarczyk equation to solve for the required stone size, and inserting the recommended values for riprap with a specific gravity of 2.65 and a fresh water specific gravity of 1.0 yields the following equation for sizing rock riprap for wave attack:

$$d_{50} \geq \frac{2}{3}\left(\frac{H_s \xi^{0.5}}{1.64\cos\theta}\right)$$

For salt water locations (specific gravity = 1.03), substitute 1.57 for 1.64 into the denominator of the above equation.

Using standard sizes the appropriate gradation can be achieved by selecting the next larger size class, thereby creating a slightly over-designed structure, but economically a less expensive one. For example, if a riprap sizing calculation results in a required d_{50} of 16.8 inches, Class V riprap should be specified because it has a nominal d_{50} of 18 inches. See Table 873.3A.

Worked examples of the Pilarczyk and the Hudson method are presented in HEC 23, Design Guideline 17. Compared with the Hudson method, the Pilarczyk method is more complicated and includes the consideration of wave period, storm duration, clearly-defined damage level and permeability of structure. The choice of the appropriate formula is dependent on the design purpose (i.e. preliminary design or detailed design).

(3) Design Height – The recommended vertical extent of riprap for wave attack includes consideration of high tide elevation, storm surge, wind setup, wave height, and wave runup. Details can be found in HEC 25, Volume 1, and HEC 23, Volume 2, Index 17.3.2.

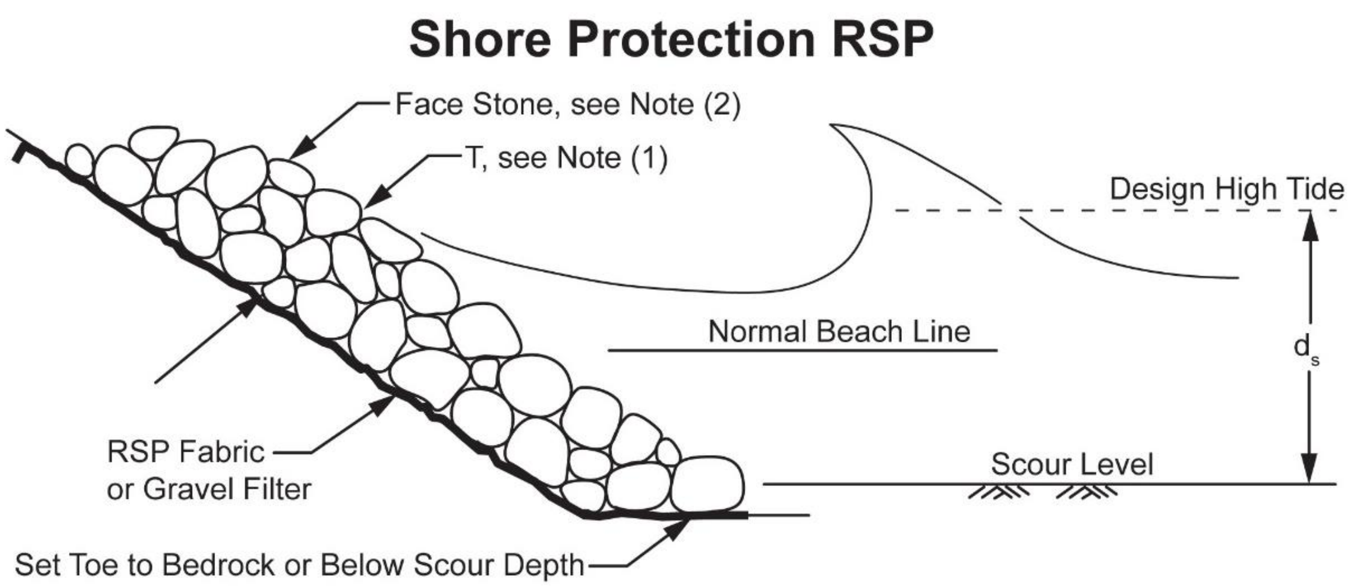
(3) *Bulkheads.* The bulkhead types are steep or vertical structures, like retaining walls, that support natural slopes or constructed embankments which include the following:

- Gravity or pile supported concrete or masonry walls.
- Sheet piling

(a) Concrete or Masonry Walls. The expertise and coordination of several engineering disciplines is required to accomplish the development of PS&E for concrete walls serving the dual purpose of slope protection and support. The Division of Structures is responsible for the structural integrity of all retaining walls, including bulkheads.

Figure 883.2I

Rock Slope Protection



NOTES:

(1) Thickness "T" = 1.5 d_{50}

(2) Face stone size is determined from Index 883.3(2)(b).

(3) RSP fabric not to extend more than 20 percent of the base width of the Mounded Toe past the Theoretical Toe.

(b) Sheet Piling. Timber, concrete and steel sheet piling are used for bulkheads that depend on deep penetration of foundation materials for all or part of their stability. High bulkheads are usually counterforted at upper levels with batter piles or tie back systems to deadmen. Any of the three materials is adaptable to sheet piling or a sheathed system of post or column piles.

Excluding structural requirements, design of pile bulkheads is essentially as follows:

- Recognition of foundation conditions suitable to or demanding deep penetration. Penetration of at least 15 feet below scour level, or into soft rock, should be assured.
- Choice of material. Timber is suitable for very dry or very wet climates, for other situations economic comparison of preliminary designs and alternative materials should be made.
- Determination of line and grade. Fairly smooth transitions with protection to high-water level should be provided.

(4) *Sea Walls.* Sea walls are structures, often concrete or stone, built along a portion of a coast to prevent erosion and other damage by wave action. Seawalls can be rigid structures or rubble-mound structures specifically designed to withstand large waves. Often they retain earth against the shoreward face. A seawall is typically more massive and capable of resisting greater wave forces than a bulkhead. Index 6.1 of HEC 25, Volume 1 provides several examples of seawall designs.

(5) *Groins.* A groin is a relatively slender barrier structure usually aligned to the primary motion of water designed to trap littoral drift, retard bank or shore erosion, or control movement of bed load.

These devices are usually solid; however, upon occasion to control the elevation of sediments they may be constructed with openings. Groins typically take the following forms of construction:

- Rock mound.
- Concreted-rock dike.
- Sand filled plastic coated nylon bags.
- Single or double lines of sheet piling.

The primary use of groins is for ocean shore protection. When used as stream channel protection to retard bank erosion and to control the movement of streambed material they are normally of lighter construction than that required for shore installation.

In its simplest or basic form, a groin is a spur structure extending outward from the shore over beach and shoal. A typical layout of a shore protection groin installation is shown in Figure 883.2J.

Assistance from the U.S. Army Corp of Engineers is necessary to adequately design a slope protection groin installation. For a more complete discussion on groins, designers should consult Volume II, Chapter 6, Section VI, of the Corps' Shore

Protection Manual until Part VI of the Coastal Engineering Manual is published. Preliminary studies can be made by using basic information and data available from USGS quadrangle sheets, USC & GS navigation charts, hydrographic charts on currents for the Northeast Pacific Ocean and aerial photos of the area.

Factors pertinent to design include:

(a) Alignment. Factors which influence alignment are effectiveness in detaining littoral drift, and self-protection of the groin against damage by wave action.

A field of groins acts as a series of headlands, with beaches between each pair aligned in echelon, that is, extending from outer end of the downdrift groin to an intermediate point on the updrift groin, see Figure 883.2K. The offset in beach line at each groin is a function of spacing of groins, volume of littoral drift, slope of sea bed and strength of the sea, varying measurably with the season. Length and spacing must be complementary to assure continuity of beach in front of a highway embankment.

A series of parallel spurs normal to the beach extending seaward would be correct for a littoral drift alternating upcoast and downcoast in equal measure. However, if drift is predominantly in one direction the median attack by waves contributes materially to the longshore current because of oblique approach. In that case the groin should be more effective if built oblique to the same degree. Such an alignment will warrant shortening of the groin in proportion to the cosine of the obliquity, see Figure 883.2K.

Conformity of groin to direction of approach of the median sea provides an optimum ratio of groin length to spacing, and the groin is least vulnerable to storm damage. Attack on the groin will be longitudinal during a median sea and oblique on either side in other seas.

(b) Grade. The top of groins should be parallel to the existing beach grade. Sand may pass over a low barrier. The top of the groin should be established higher than the existing beach, say 2 feet as a minimum for moderate exposure combined with an abundance of littoral drift, to 5 feet for severe exposure and deficiency of littoral drift.

The shore end should be tapered upward to prevent attack of highway embankment by rip currents, and the seaward end should be tapered downward to match the side slope of the groin in order to diffuse the direct attack of the sea on the end of the groin.

(c) Length and Spacing. The length of groin should equal or exceed the sum of the offset in shoreline at each groin plus the width of the beach from low water (LW) to high water (HW) line, see Figure 883.2I. The offset is approximately the product of the groin spacing and the obliquity (in radians) of the entrapped beach. The width of beach is the product of the slope factor and the range in stage. The relation can be formulated:

$L = ab + rh$

Where:

L = Length of groin, feet

a = obliquity of entrapped beach in radians

b = beach width between groins, feet

r = reciprocal of beach slope

h = range in stage, feet

For example, with groins 400 feet apart, obliquity up to 20 degrees, on a beach sloping 10:1 with a tidal range of 11 feet,

$$L = 0.35 \times 400 + 10 \times 11 = 250\, feet$$

The same formula would have required L = 390 feet for 800-foot spacing, reducing the aggregate length of groins but increasing the depth of water at the outer ends and

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