CALIFORNIA
DEPARTMENT OF TRANSPORTATION

Manual Change Transmittal

TITLE	APPROVED BY	NO. 5
HIGHWAY DESIGN MANUAL SEVENTH EDITION – MCT #5	Eric Souza, Chief Acting for Eric : Pyo Hong	Date Issued: 08/05/25 Page 1 of 6
SUBJECT AREA Table of Contents, Chapters 40, 100, 800, 820, 830, 840, 850, 860, 870, 880, and 890	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 and Chapters 40, 100, 800, 820, 830, 840, 850, 860, 870, 880, 890, and Index of the Seventh Edition, Highway Design Manual (HDM) have been revised. The Manual Change Transmittal including associated changed pages of the HDM is available on the Department's Division of Design Website at:

https://dot.ca.gov/programs/design/manual-highway-design-manual-hdm

Changes include updates related to federal funding, design period, wildlife crossing, plastic pipe, rock slope protection, sea level rise, permit requirements, and best management practices. Also included are: website updates, clarification language, typographical corrections, reference corrections, and updates to figures, index numbers, and tables.

These changes are effective August 8, 2025 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 website indicated above. 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.

A summary of significant revisions refenced in this Manual Change Transmittal are as follows:

Index 41.1 **General , Page 40-1**
Update to Federal-Aid funding to include Moving Ahead for Progress in the 21st Century Act (MAP-21) and Infrastructure Investment and Jobs Act (IIJA).

Index 42.2 **Interstate, Page 40-2**
Update related to new federal law and regulations regarding funding for Federal-aid highway projects as well as access control.

Index 43.1 **Surface Transportation Block Grant Program (STBG) , Page 40-2**
Update to STBG guidance and funding apportion as a result of the Fixing America's Surface Transportation (FAST) Act.

Index 43.3 **Bridge Replacement and Rehabilitation Program, Page 40-3**
Update to Bridge Formula Program (BFP) Implementation Guidance and use of funds provided under this program.

Index 43.4 **Federal Lands Highway Program, Page 40-3**
Update to guidance and programs related to FAST act and Infrastructure Investment and Jobs Act (IIJA).

Index 43.5 **Highway Safety Improvement Program, Page 40-3**
Update to HSIP guidance with references to federal regulations.

Index 43.6 **Emergency Relief, Page 40-4**
Update regarding legislation and eligibility requirements for the repair or reconstruction of Federal-aid highways and roads on Federal lands, which suffer serious damage as a result of natural disaster or catastrophic failures from external causes.

Index 44.1 **Funding Eligibility, Page 40-4**
Updates include changes to roles and responsibilities, clarification of the process for determining federal program eligibility, and additional information about funding thresholds and requirements. The final approval authority for requesting federal participation was updated.

Index 44.2 **Federal Participation Ratio, Page 40-4 to 40-5**
Update to "Federal share" based on related statutory provisions and contact for determination of federal funding and proposed federal reimbursement rates.

Index 45.1	**California Stewardship and Oversight Agreement with FHWA, Page 40-5** Addition of new Index containing information previously HDM found in Index 43.2, with an update to include new office contact as well as a discussion on alternative procurement processes such as Construction Manager General Contractor and Design Build.
Index 103.2	**Design Period , Page 100-6** Update includes the addition of guidance for determining design period for non-interstate projects (excluding roundabouts) to allow for greater flexibility in choosing design period consistent with project scopes.
Chapter 800 820, 830,840 850, 860,870 880, 890	**Chapter 800 series, Various Pages** Update to text, figures, and table for clarity and office contact information.
Topic 807	**Selected Drainage References, Page 800-46** Update to associated indexes to include new references.
Table 808.1	**Summary of Related Computer Programs & Web Applications, Page 800-51** Update to include SMS/SRH-2D.
Index 821.1	**Introduction, Page 820-1** Update of guidance for wildlife connectivity.
Index 821.6	**Wildlife Passage Consideration, Page 820-37** New Index to provide guidance on wildlife passage consideration and references in compliance with SHC 158.
Index 825.3	**Computer Programs, Page 820-42** Update to incorporate SMS/SRH-2D program.
Topic 827	**Outlet Design, Page 820-45** Update to indexes to include new references.
Index 835.2	**Earth Berms, Page 830-11** Addition of new guidance and references for Earth Berm designs.
Index 838.2	**Design Criteria, Page 830-22** Update to references.

Index 842.6 **Design Service Life, Page 840-5**
Update to remove Bituminous coating as protective coating options from standards.

Index 853.6 **Invert Paving with Concrete, Page 850-12**
Update to include Special Provision.

Index 855.5 **Material Susceptibility to Fire , Page 850-40 to 41**
Update to provide reference Cal Fire's "Fire Hazard Severity Zone Map".

Index 856.4 **Plastic Pipe, Page 850-43**
Update include adding guidance on managing conditions when the water table is to prevent pipe flotation with recommended precautionary measures

Table 856.3A **Corrugated Steel Pipe Helical Corrugations, Page 850-44**
Update to values for Maximum Height of Cover.

Table 856.3C **Corrugated Steel Pipe 2⅔" x ½" Annular Corrugations, Page 850-46**
Update to values for Maximum Height of Cover.

Index 861.12 **References, Page 860-6**
Update to references.

Table 865.2 **Permissible Shear and Velocity for Selected Lining Materials(2) (cont.) , Page 860-21**
Update to Rock Slope Protection Boundary Type and values for Permissible Shear Stress and Permissible Velocity.

Index 871.3 **Selected References , Page 870-3 to 4**
Update to references.

Index 872.1 **Planning , Page 870-4**
Update to FHWA's HEC's references.

Index 872.3 **Geomorphology and Site Consideration, Page 870-6**
Update to guidance for river morphology and river response.

Index 873.2 **Design High Water and Hydraulics, Page 870-30 to 31**
Update to guidance for discussion on the fundamentals of alluvial channel flow.

Index 873.3 **Armor Protection, Page 870-31 to 54**
Update to references for Bridge Scour and Stream Instability Countermeasures, Design Guideline.

Index 873.6 **Coordination with Division of Engineering Services and Structures Maintenance and Investigations , Page 870-62 to 64**
Update to include discussion on countermeasures design at bridge abutments and piers. Embankment constructability impact nearby structures guidance.

Index 881.2 **Design Philosophy , Page 880-1 to 2**
Update to refences and nature-based coastal protection.

Index 881.3 **Selected References , Page 880-2 to 3**
Update to existing references and addition of new references.

Index 883.2 **Design High Water, Design Wave Height and Sea level Rise , Page 880-6 to 22**
Update to guidance for design high water level, design wave heights, and method for evaluating sea-level rise.

Table 883.1A **Crescent City Example Comparison for 2060 , Page 880-20**
Update to values to reflect new guidance updates.

Table 883.1B **Sea-level scenarios Crescent City , Page 880-20**
Update values to reflect new guidance updates.

Index 883.3 **Coastal Protection , Page 880-23 to 31**
Update guidance to include nature-based hybrid strategies with new references.

Index 891.1 **Introduction , Page 890-1**
Update to information on National Pollutant Discharge Elimination System (NPDES) permits for stormwater management.

Index 891.3 **Permits , Page 890-2 to 3**
Update to include new guidance on NPDES permit.

Index 891.4 **Design Standards , Page 890-3 to 4**
Update to new guidance on appropriate standard plans and standard specifications to comply with permit requirements.

Index 892.1 **General , Page 890-4**
Update to stormwater runoff volumes for sizing BMPs.

Index 892.2 **Types of Strategies , Page 890-4 to 6**
Updates to stormwater management strategies and BMPs for Design, Construction and Maintenance as well as Sustainability Groundwater Management Act requirements.

Index 892.3 **Design Considerations , Page 890-7**
Update to include additional considerations for stormwater management strategies.

Index 893.1 **General , Page 890-9**
Update to include the addition of references and additional guidance on Maintenance BMPs.

Enclosures: Table of Contents, HDM Chapter 40, Index 103.2, Chapter 800, 820, 830, 840, 850, 860, 870, 880, and 890.

Table of Contents

CHAPTER 40 – FEDERAL-AID FUNDING

Topic 41 – Enabling Legislation

Index 41.1 – General

The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 was the first major transportation legislation since the Interstate System was enacted.

ISTEA changed the established Federal-Aid system. During the 20 years prior to ISTEA there were four Federal-Aid systems: Interstate, Primary, Secondary, and Urban. Now under ISTEA, instead of four Federal-aid systems there are two, the National Highway System (NHS) and the Interstate System, which is a component of the National Highway System.

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

The Moving Ahead for Progress in the 21st Century Act (MAP-21) was signed into law in 2012 and streamlined the performance-based surface transportation program, establishing asset management as a key direction for transportation funding. In 2015, the Fixing America's Surface Transportation (FAST) Act was signed into law, providing the first federal law in over a decade to provide long-term funding for infrastructure planning and investment. The FAST Act maintained focus on safety, keeping intact the established structure of the various highway programs, continued efforts to streamline delivery, and for the first time, provided dedicated funding for freight projects.

In November of 2021, the Infrastructure Investment and Jobs Act (IIJA) was signed into law. The IIJA is the largest long-term infrastructure investment in our Nation's history, providing federal investments in infrastructure including: roads, bridges, mass transit, water infrastructure, resilience, and broadband. In addition to our core federal aid programs, IIJA created new programs, including Promoting Resilient Operations for Transformative, Efficient, and Cost-Saving Transportation (PROTECT), Carbon Reduction Program (CRP), and the Bridge Formula Program (BFP), ensuring dedicated funding for specific scopes of work.

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

Topic 42 – Federal-Aid System

42.1 National Highway System

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

42.2 Interstate

As a result of ISTEA, the Interstate Highway System is a part of the NHS, but retains its separate identity. As a condition of funding for Federal-aid highway projects, Federal law prohibits State departments of transportation from adding any point of access to or from the Interstate System without the approval of the Secretary of Transportation and all new or modified points of access must be approved by FHWA and developed in accordance with federal laws and regulations as specified in 23 U.S.C. 109 and 111, 23 C.F.R. 624 and 625.4, and 49 C.F.R. 1.48(b)(1). Revenue from the Federal gas and other motor-vehicle user taxes was credited to the Highway Trust Fund to pay the Federal share of Interstate and all other Federal-aid highway projects. In this way, the Act guaranteed construction of all segments on a "pay-as-you-go" basis, where the program is self-financing without contributing to the Federal budget deficit.

Topic 43 – Federal-Aid Programs

43.1 Surface Transportation Block Grant Program (STBG)

The Surface Transportation Block Grant (STBG) Program provides flexible funding that may be used by States and localities for projects to preserve and improve the conditions and performance on any Federal-aid highway, bridge, tunnel projects on any public road, pedestrian and bicycle infrastructure, and transit capital projects, including intercity bus terminals. These roads are collectively referred to as Federal-aid roads.

As under the FAST Act, the IIJA directs FHWA to apportion funding as a lump sum for each State then divide that total among apportioned programs. The IIJA requires the Secretary of Transportation to set aside 10 percent of STBG funds for Transportation Alternatives. Additionally, it requires that 2 percent of State's STBG apportionment be allocated for State Planning and Research (SPR), and an amount equal to at least 20 percent of the State's FY 2009 Highway Bridge Program apportionment for use on certain types of projects related to bridges and low water crossings. In addition, 55 percent of a State's STBG apportionment is to be suballocated in specific population-based areas and the remaining 45 percent obligated in any area of the State.

43.2 Congestion Mitigation and Air Quality Improvement Program (CMAQ)

The Congestion Mitigation and Air Quality (CMAQ) Improvement Program directs funds toward transportation projects in Clean Air Act non-attainment areas for ozone, carbon monoxide and particulate matter (both PM10 and PM2.5). Projects using CMAQ funds contribute to meeting the attainment of national ambient area air quality standards. CMAQ funds may not be used for projects which will increase capacity for single occupant vehicles. Exceptions might include HOV lanes which allow single occupant vehicles at other than peak travel times or auxiliary lanes.

43.3 Bridge Replacement and Rehabilitation Program

FHWA provides guidance for the implementation of a bridge replacement and rehabilitation program authorized in the Department of Transportation Appropriations Act, which clarifies eligible projects and provides information on the Administration's priorities and the use of funds. Additionally, please refer to the Bridge Formula Program (BFP) Implementation Guidance or guidance on Administration priorities and use of funds such as those provided under this program.

43.4 Federal Lands Highway Program

The Federal Lands Highway Program is administered by the FHWA Office of Federal Lands Highway. This program was established by the FAST Act and has been continued under the Infrastructure Investment and Jobs Act. Caltrans receives funding from the FHWA Office of Federal Lands Highway through programs like the Federal Lands Access Program (FLAP). This program provides funds for projects that improve transportation facilities providing access to, adjacent to, or located within Federal lands.

43.5 Highway Safety Improvement Program

The Highway Safety Improvement Program (HSIP) is a core Federal-aid program with the purpose to achieve a significant reduction in traffic fatalities and serious injuries on all public roads, including non-State-owned roads and roads on tribal land. The HSIP requires a data-driven and strategic approach to improving highway safety on all public roads with a focus on performance. The HSIP is legislated under Section 148 of Title 23, United States Code (23 U.S.C. 148) and regulated under Part 924 of Title 23, Code of Federal Regulations (23 CFR Part 924). The HSIP consists of three main components: the Strategic Highway Safety Plan (SHSP), State HSIP or program of highway safety improvement projects, and the Railway-Highway Crossing Program (RHCP).

Current programming of HSIP-eligible projects includes both SHOPP 20.xx.201.010 and 20.xx.201.015 programs. Additionally, the IIJA established a new Special Rule under the Highway Safety Improvement Program under section 148 of title 23 of the United States Code (U.S.C.) for vulnerable road user (VRU) safety and continued the two existing special rules for High-Risk Rural Roads (HRRR) and Older Drivers and Pedestrians without change. The VRU Special Rule is part of a larger focus on non-motorist safety that includes a new requirement for States to complete VRU safety assessments (23 U.S.C. 148(l)).

43.6 Emergency Relief

Congress authorized in Title 23, United States Code, Section 125, a special program from the Highway Trust Fund for the repair or reconstruction of Federal-aid highways and roads on Federal lands, which have suffered serious damage as a result of (1) natural disasters or (2) catastrophic failures from an external cause. This program, commonly referred to as the Emergency Relief (ER) program, supplements the commitment of resources by States, their political subdivisions, or other Federal agencies to help pay for unusually heavy expenses resulting from extraordinary conditions.

In order for a project to be eligible under the ER program, the project must have an FHWA-approved Damage Assessment Form (DAF) typically provided by the District Maintenance Major Damage Restoration Engineer (MDRE). The State programs ER-eligible projects under either the SHOPP Emergency Opening 20.xx.201.130 program or the Permanent Restoration 20.xx.201.131 program. Emergency Opening projects may be funded at 100 percent for the first 270 days of construction, however, a final determination will be made by the Office of Federal Resources based on the scope of work and inclusion of permanent repairs under emergency opening procedures.

The project manager should work closely with the District MDRE to establish eligibility under the ER Program based on the FHWA-approved DAF. Furthermore, the project manager should be aware of the high priority of ER-funded projects, based on the need to meet a shortened 2-year Construction authorization timeline. See the Project Development Procedures Manual, Chapter 9, Article 5 for additional information regarding the acceleration and high priority of ER-funded projects.

Topic 44 – Funding Determination

44.1 Funding Eligibility

Each Federal program has certain criteria and requirements. During design, the project manager is to consult with the HQ Office of Federal Resources (OFR) to determine the appropriate Federal program each individual project is eligible for and the level of future Federal involvement. The HQ OFR Area Engineer can assist with a determination of whether or not a SHOPP or STIP project is qualified for federal funding, based on the current Federal Aid Project Funding Guidelines, which establish federal funding thresholds based on Construction or Right of Way capital costs, among other aspects. FHWA Major Projects \$500m or greater and mini-Major Projects \$100m or greater have additional requirements such as Project Management Plans and Financial Plans, see the Project Development Workflow Guide, Task D408b for additional information. The final determination to request Federal participation will be made by the Office of Federal Resources.

44.2 Federal Participation Ratio

The maximum share of project cost that may be funded with Federal-aid highway funds (the “Federal share”) varies based upon the Federal-aid program from which the project receives funding. In some cases, the Federal share is also adjusted based on related statutory provisions. The typical federal share under the sliding scale is 91.57 percent on Interstate projects and 88.53 percent on non-Interstate projects for federally eligible roads. Certain specified types of projects, mostly targeting safety improvements, are eligible to receive a Federal share of 100 percent. A toll project under 23 U.S.C. 129 is eligible for a maximum Federal share of 80 percent (regardless of whether the project would have qualified for a higher Federal share if advanced as a non-toll facility). The project manager should work directly with the Office of Federal Resources to determine federal funding and proposed federal reimbursement rates.

Topic 45 – FHWA Stewardship and Oversight

45.1 California Stewardship and Oversight Agreement with FHWA

The goal under the Stewardship and Oversight Agreement is to document the roles and responsibilities of the FHWA's California Division Office and Caltrans with respect to project approvals and related responsibilities, and to document the methods of oversight which will be used to efficiently and effectively deliver the Federal-aid Highway Program. The FHWA's Risk-Based Project Involvement process combines risks, data, and judgement to select risk-based involvement projects (RBI projects) and develop stewardship and oversight activities beyond what is required. The FHWA selects RBI projects statewide bi-annually. The Department generally still retains approval authority over actions on RBI projects. However, there are some exceptions for example on projects using alternative procurement processes such as Construction Manager General Contractor and Design Build, where the approval of actions may be retained by the FHWA. FHWA will verify compliance with federal regulations via annual program and process reviews. See the Project Development Procedures Manual for other essential procedures regarding the Stewardship and Oversight Agreement between the Department and FHWA. For additional information see the FHWA webpage on Stewardship and Oversight. See the Department Design website for the current Stewardship and Oversight Agreement between FHWA California Division Office and Caltrans.

(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.

Design period for all other types of projects, except roundabouts, should be determined based on the project's scope, acceptable level of service, future land usage, community development, and other existing operational characteristics in the area. Design period should be evaluated on a case-by-case basis with the concurrence by the Project Delivery Coordinator or the District Design Liaison.

For roundabout design period guidance, see Index 405.10.

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 hydrology and hydraulics reports and texts have been used to compile this highway drainage guide. Frequent references are made to these publications. Where there is a conflict in information or procedure, engineers must look at all pertinent parameters and use their best judgment, to determine which approach is the most consistent with the objectives of Caltrans drainage design principles and which most closely relates to the specific design problem or project.

Topic 802 – Drainage Design Responsibilities

802.1 Functional Organization

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

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

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

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

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

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

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

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

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

(b) The structural adequacy of all drainage facilities.

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

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

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

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

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

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

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

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

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

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

(f) Compliance with Federal-Aid Policy Guide, Transmittal 1, G 6012.1 for storm drain systems.

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

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

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

- Storm drain design and calculations.
- Drainage basins exceeding 320 acres.
- Hydrograph development or routing.
- Open channel modification or realignment.
- Retention or detention basins.
- Backwater analysis.
- High potential for flood damage litigation.

- Scour analysis or sediment transport (typically forwarded to DOS).
- Culvert designs greater than 36 inches in diameter.
- Encroachments on FEMA designated floodplains.
- Modifications to inlet or outlet capacities on existing culverts or drainage inlets (e.g., placement of safety end grates, conversion of side opening inlets to grated inlets, etc.).
- Unique hydraulic design features (e.g., energy dissipator design, pumping stations, siphons, etc.).

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

(5) *Materials Engineering and Testing Services.* METS provides advice and guidance to other Caltrans Offices and Branches concerning service life, physical properties, and structural adequacy of materials used in drainage design.

802.2 Culvert Committee

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

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

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

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

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

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

The authority of the Committee is advisory only, and recommendations of the Committee are submitted to the Chief, Division of Design for approval and implementation through design guidelines and standards.

Requests for consideration of new materials, methods, or procedures should be directed to the Committee Chairman.

802.3 Bank and Shore Protection Committee

The Caltrans Bank and Shore Protection Committee is composed of representatives from DES Structures Maintenance and Investigation, Office of Hydraulics and Stormwater Design, METS, Division of Construction, and Division of Maintenance. It is chaired by the Office of Hydraulics and Stormwater Design representative. The Committee performs the following functions:

(a) Acts as a service and an advisory group available to Districts and Caltrans Offices and Branches upon written request for special investigations or study. Requests for special investigation of rock slope protection, channel or bridge protection, major channel changes, etc. should be directed to the Committee Chair.

(b) Provides conceptual input and acts as approval authority for supplements or modifications to bank and shore protection practice publications as warranted.

(c) Investigates and provides input toward the development of detailed design criteria for the various types of bank and shore protection.

(d) Observes performances of existing and/or experimental installations during or following severe exposures. The Districts or Caltrans Offices or Branches are requested to inform the Chair, Bank and Shore Protection Committee, or any available members of the Committee, of damage to installations by flood or high seas.

(e) Upon submission by the Department's New Products Coordinator, the Committee evaluates new products and processes related to bank and shore protection for possible approval.

Topic 803 – Drainage Design Policies

803.1 Basic Policy

In drainage design, the basic consideration is to protect the department’s facilities against damage from storm and subsurface waters, taking into account the effect of the proposed improvement on travelers, property, and wildlife. Unless the State would benefit thereby, or the cost is borne by others, no improvement in the drainage of areas outside the right of way is to be considered on Caltrans projects.

803.2 Cooperative Agreements

The extent of the department's financial participation in cooperative drainage improvement projects must be commensurate with the benefits to the Department and the traveling public.

(1) *Local Agencies.* Caltrans may participate with Local Agencies, Flood Control Districts or Drainage Assessment Districts on drainage improvement projects. Such projects must be covered by a formal agreement prepared and processed in accordance with instructions in the Caltrans Cooperative Agreement Manual.

(2) *Federal and State Flood Control Projects.* The cost of upgrading or modifying existing State highway facilities to accommodate Federal and/or State funded flood control projects is normally the responsibility of the agency funding the project. As necessary, Caltrans may enter into agreements containing provisions that the cost of betterments to existing highways, including drainage features, will be paid for by the Department. The Cooperative Agreement Manual contains procedures for preparing interagency agreements.

803.3 Up-Grading Existing Drainage Facilities

(1)*Rehabilitation and Reconstruction Projects.* The hydraulic adequacy, as well as the structural adequacy of existing drainage facilities should be evaluated early in the project development process on pavement rehabilitation and highway reconstruction projects

804.2 Authority

Title 23, CFR, Part 650, Subpart A, prescribes FHWA's "... policies and procedures for the location and hydraulic design of highway encroachments on floodplains, ..." The CFR's may be found on-line at the National Archive's Code of Federal Regulations eCFR tool at: https://www.ecfr.gov/

804.3 Applicability

The guidance provided herein establishes Caltrans procedures whenever a floodplain encroachment is anticipated. Adherence to these procedures will also ensure compliance with applicable Federal regulations which apply to any Federally approved highway construction, reconstruction, rehabilitation, repair, or improvement project which affects the (100-year) base floodplain. Work outside the limits of the base floodplain should be reviewed to see if it affects the (100-year) base floodplain. The only exception is repairs made with emergency funds during or immediately following a disaster. The premise is that all Federal-aid projects be evaluated and that diligent efforts be made to:

- Avoid significant floodplain encroachments where practicable.
- Minimize the impact of highway actions that adversely affect the base floodplain.
- Be compatible with the National Flood Insurance Program (NFIP) of the Federal Emergency Management Agency (FEMA).

804.4 Definitions

The following definitions of terms are made for the purpose of uniform application in the documentation and preparation of floodplain evaluation reports. Refer to Title 23, CFR, Part 650, Section 650.105 for a complete list of definitions.

(1) *Base Flood.* The flood or tide having a 1 percent chance of being exceeded in any given year (100-year flood).

(2) *Base Floodplain.* The area subject to flooding by the base flood. Every watercourse (river, creek, swale, etc.) is subject to flooding and theoretically has a base floodplain.

(3) *Design Flood*. The peak discharge, volume if appropriate, stage or wave crest elevation of the flood associated with the probability of exceedance selected for the design of a highway encroachment. By definition, the highway will not be inundated from the stage of the design flood.

(4) *Encroachment.* An action within the limits of the base floodplain. Any construction activity (access road, building, fill slopes, bank or slope protection, etc.) within a base floodplain constitutes an encroachment.

(5)Location Hydraulic Study. A term from 23 CFR, Section 650.111 referring to the preliminary investigative study to be made of base floodplain encroachments by a proposed highway action. The extent of investigation and the discussion content in the required documentation of the "Location Hydraulic Study" is very site specific and need be no more than that which is commensurate with the risk(s) and impact(s) particular to the location under consideration. The information developed, documented (refer to Figure 804.7A) and retained in the project file is the suggested minimum necessary for compliance.

The National Flood Insurance Act of 1968, as amended (42 U.S.C. 4001-4127) requires that communities adopt adequate land use and control measures to qualify for insurance. To implement this provision, the following Federal criteria contains requirements which may affect certain highways:

- In riverine situations, when the Administrator of the Federal Insurance Administration has identified the flood prone area, the community must require that, until a floodway has been designated, no use, including land fill, be permitted within the floodplain area having special flood hazards for which base flood elevations have been provided, unless it has been demonstrated that the cumulative effect of the proposed use, when combined with all other existing and reasonably anticipated uses of similar nature, will not increase the water surface elevation of the 100-year flood more than 1 foot at any point within the community.
- After the floodplain area having special flood hazards has been identified and the water surface elevation for the 100-year flood and floodway data have been provided, the community must designate a floodway which will convey the 100-year flood without increasing the water surface elevation of the flood more than 1 foot at any point and prohibit, within the designated floodway, fill, encroachments and new construction and substantial improvements of existing structures which would result in any increase in flood heights within the community during the occurrence of the 100-year flood discharge.
- The participating cities and/or counties agree to regulate new development in the designated floodplain and floodway through regulations adopted in a floodplain ordinance. The ordinance requires that development in the designated floodplain be consistent with the intent, standards and criteria set by the National Flood Insurance Program.

804.11 Coordination with FEMA

There should be Caltrans coordination with FEMA in situations where administrative determinations are needed involving a regulatory floodway or where flood risks in NFIP communities are significantly impacted. The circumstances which would ordinarily require coordination with FEMA include the following.

- When a proposed crossing encroaches on a regulatory floodway and, as such, would require an amendment to the floodway map.
- When a proposed crossing encroaches on a floodplain where a detailed study has been performed but no floodway designated and the maximum 1 foot increase in the base flood elevation would be exceeded.
- When a local community is expected to enter into the regular program within a reasonable period and detailed floodplain studies are under way.
- When a local community is participating in the emergency program and the base FEMA flood elevation in the vicinity of insurable buildings is increased by more than 1 foot. Where insurable buildings are not affected, it is sufficient to notify FEMA of changes to the base flood elevations as a result of highway construction.
- The draft (EIS/EA) should indicate the NFIP status of affected communities, the encroachments anticipated and the need for floodway or floodplain ordinance amendments. If a determination by FEMA would influence the selection of an alternative, a commitment from FEMA should be obtained prior to the final environmental impact Statement (FEIS) or FONSI.

More information regarding FEMA can be found on-line at FEMA's Flood Insurance website: http://www.fema.gov/nfip/.

FEMA has developed a comprehensive listing of all numerical models that are accepted for NFIP usage. These models can be accessed online at FEMA's Software for Flood Mapping website: https://www.fema.gov/flood-maps/software

Topic 805 – Preliminary Plans

805.1 Required FHWA Approval

Current Federal policy requires the review and approval of plans for unusual structures. (See Indices 805.2 - 805.6) by FHWA. FHWA will no longer review and approve major structures (those with greater than 125,000 square feet of deck area) or pumping plants with greater than 20 CFS design discharge. Submittal of plans for unusual structures for review applies only to new construction on the Interstate system. The responsibility for the oversight of unusual structures on other Federal-aid and non-Federal-aid highways will be assumed by the state.

Federal review and approval may take place at either their Division Office or FHWA Headquarters in Washington, D.C. Early submission of necessary data is critical in order to receive a timely approval.

805.2 Bridge Preliminary Report

A Bridge Preliminary Report will be prepared by Structures Design, in the Division of Engineering Services and submitted to the California FHWA Division Office in Sacramento for approval of unusual bridges and structures.

An unusual bridge involves difficult or unique foundation problems, new or complex designs involving unique design or operational features, longer than normal spans or bridges for which the design procedures depart from current acceptable practice. Examples include cable stayed, suspension, arch, segmental concrete bridges, trusses and other bridges which deviate from AASHTO Standard Specifications or Guide Specifications for Highway Bridges, bridges requiring abnormal dynamic analysis for seismic design, bridges designed using a three-dimensional computer analysis, bridges with spans exceeding 500 feet, and bridges which include ultra high strength concrete or steel.

805.3 Storm Drain Systems

The District will submit preliminary plans and hydraulic data for unusual storm drain systems to the California FHWA Division Office in Sacramento for storm drain systems that carry more than 200 CFS or have an accumulated surface detention storage system of more than five acre-feet.

805.4 Unusual Hydraulic Structures

The District will submit preliminary plans and hydraulic data for unusual hydraulic structures to the California FHWA Office in Sacramento. For projects on the interstate system, FHWA Headquarters Office of Bridge Technology approval is required for hydraulic structures involving unusual stream stability countermeasures or unique design techniques. The Division of Engineering Services will submit preliminary plans and hydraulic data to the California FHWA Division Office in Sacramento for unusual structures such as tunnels, complex or unique geotechnical structures and complex or unique hydraulic structures.

805.5 Levees and Dams Formed by Highway Fills

The District will submit preliminary plans and other supportive data to the California FHWA Division Office in Sacramento for approval of:

(a) Highway fills which will function as a levee and serve the purpose of reducing the flooding of adjacent areas.

(b) Dams formed by highway fills which will permanently impound water more than 25 feet in depth or 50 acre-feet in volume. See Index 829.9 Dams, for legal definition of a dam and regulations relative to approval by the California Department of Water Resources. Also see 23 CFR 650.115(c) for the definition of design standards regarding the use of highway fills as dams.

805.6 Geotechnical

The District shall submit preliminary plans and technical data for major or unusual geotechnical features to the California FHWA Division Office for approval. Major geotechnical features include unusually deep cuts or high fills where the site geology is potentially unstable, landslide corrections, and large retaining walls (cantilever, permanent ground anchor, and soil reinforcement). FHWA Headquarters Bridge Division approval is required for unusual geotechnical features, such as new or complex retaining wall systems or ground improvement systems.

805.7 Data Provided by the District

The following items of supportive information must be provided with requests for FHWA approval:

(a) Preliminary plans and profiles:

- Approach layouts.
- Drainage plans.

(b) Hydraulic design studies:

- Design Q and frequency.
- Hydraulic grade lines.
- Inflow - Outflow hydrographs.
- Capacity of reservoirs or pump storage systems.
- Pump capacity.

Weir. A low overflow dam or sill for measuring, diverting, or checking flow.

Well. (1) Artificial excavation for withdrawal of water from underground storage. (2) Upward component of velocity in a stream.

Wetland. Those areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas.

Windbreak. Barrier fence or trees to break or deflect the velocity of wind.

Windwave. A wave generated and propelled by wind blowing along the water surface.

Young. Immature, said of a stream on a steep gradient actively scouring its bed toward a more stable grade.

Topic 807 – Selected Drainage References

807.1 Introduction

Hydraulic and drainage related reference publications listed are grouped as to source.

807.2 Federal Highway Administration Hydraulic Publications

Current publications are available from FHWA Hydraulics Publication website: https://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm

(1) Hydraulic Engineering Circulars (HEC).

HEC No.	Title	Date	FHWA Publication
9	Debris-Control Structures	2005	IF-04-016
14	Hydraulic Design of Energy Dissipators for Culverts and Channels	2006	NHI-06-086
15	Design of Roadside Channels with Flexible Linings	2005	IF-05-114
17	Highways in the River Environment, Floodplains, Extreme Events, Risk and Resilience	2016	HIF-16-018
18	Evaluating Scour at Bridges	2012	HIF-12-003
20	Stream Stability at Highway Structures	2012	HIF-12-004
21	Bridge Deck Drainage Systems	1993	SA-92-010
22	Urban Drainage Design Manual	2024	HIF-24-006

HEC No.	Title	Date	FHWA Publication
23	Bridge Scour and Stream Instability Countermeasures	2009	NHI-09-111 NHI-09-012
24	Highway Stormwater Pump Station Design	2001	NHI-01-007
25	Highways in the Coastal Environment	2020	HIF-19-059
26	Culvert Designer Aquatic Organism Passage	2010	HIF-11-008

(2) Hydraulic Design Series (HDS).

HDS No.	Title	Date	FHWA Publication #
2	Highway Hydrology	2024	HIF-24-007
4	Introduction to Highway Hydraulics	2008	NHI-08-090
5	Hydraulic Design of Highway Culverts (GPO 050-001-00298-1)	2012	HIF-12-026
7	Hydraulic Design for Safe Bridges	2012	HIF-12-018

(3) Other Publications.

Title	Date	FHWA Publication#
FHWA Bridge Inspector's Reference Manual	2023	NHI-23-024
FHWA-FLH Culvert Assessment and Decision-Making Procedures Manual (hard copy only)	2010	CFL/TD-10-005
Culvert Pipe Liner Guide and Specifications	2005	CFL/TD-05-003
Aquatic Organism Passage at Highway Crossings: An Implementation Guide	2024	FHWA-HIF-24-054

807.3 American Association of State Highway and Transportation Officials (AASHTO)

(1) Highway Drainage Guidelines

The AASHTO Highway Drainage Guidelines were previously published as individual volumes, the 2007 edition moved to chapter designations, as follows:

1. - Hydraulic Considerations in Highway Planning and Location
2. - Hydrology
3. - Erosion and Sediment Control in Highway Construction
4. - Hydraulic Design of Culverts
5. - The Legal Aspects of Highway Drainage
6. - Hydraulic Analysis and Design of Open Channels
7. - Hydraulic Analysis for the Location and Design of Bridges
8. - Hydraulic Aspects in Restoration and Upgrading of Highways
9. - Storm Drain Systems
10. - Evaluating Highway Effects on Surface Water Environments
11. - Highways along Coastal Zones and Lakeshores
12. - Stormwater Management
13. - Training and Career Development of Hydraulic Engineers
14. - Culvert Inspection, Material Selection, and Rehabilitation
15. - Guidelines for Selecting and Utilizing Hydraulics Engineering Consultants

   Appendix. Glossary of Highway-Related Drainage Terms

The layout and chapter list are subject to change as this manual is being updated. Refer to the AASHTO website for the current edition. The current edition may be purchased through AASHTO, 444 North Capitol St., N.W., Suite 225, Washington D.C. 20001.

(2) AASHTO Drainage Manual

The AASHTO Drainage Manual (ADM) is divided into two volumes. Volume One provides states with guidelines or examples for drainage design policies, criteria, and standards. Volume Two provides hydrologic and hydraulic design procedures that are frequently used by highway hydraulics engineers.

(3) AASHTO Culvert and Storm Drain System Inspection Guide

This guide provides inspectors with methods to rate the conditions for culvert and storm drain system components.

807.4 California Department of Transportation

The following publications are available from the Caltrans website.

https://dot.ca.gov/programs/design

- Bridge Design Practice Manual
- Structure Technical Policy (STP), 2.6 Bridge Design Manual
- Manual of Test - Volumes 1, 2, and 3
- Construction Contract Standards: Standard Plans, Standard Specifications and Contract Item Codes (available online only)

807.5 U.S. Department of Interior - Geological Survey (USGS)

- Magnitude and Frequency of Floods in California - Water Resources Investigation 77-21.
- Methods for Determining Magnitude and Frequency of Floods in California, Based on Data through Water Year 2006, SIR 2012-5113
- Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States –Open-File Report 93-419.
- Guide For Determining Flood Flow Frequency - Bulletin #17C.
- Water Resources Data for California, Part 1: Surface Water Records, Volumes 1 and 2 (1965) – Water Data Report CA-65-1-1 and CA-65-1-2.
- Rock Riprap Design for Protection of Stream Channels Near Highway Structures (1987) Volumes 1 and 2 (1987). – Water Resources Investigation Reports 86-4127 and 86-4128.
- Regional Skew for California, and Flood Frequency for Selected Sites in the Sacramento-San Joaquin River Basin, Based on Data through Water Year 2006 (2011) - Scientific Investigations Report 2010-5260.

807.6 U.S. Department of Agriculture - Natural Resources Conservation Service (NRCS)

- Engineering Design Standards.
- Urban Hydrology for Small Watersheds -Technical Release 55

807.7 California Department of Water Resources

The California Department of Water Resources provides intensity, duration, and frequency data from the California Department of Water Resources network of rain gauges at the California Water Watch website: Statewide Hydroclimate and Water Supply Conditions – Precipitation at Regional Scale, at: https://cww.water.ca.gov/regionscale.

807.8 University of California - Institute of Transportation and Traffic Engineering (ITTE)

- Street and Highway Drainage - Course Notes, Volumes 1 and 2.

807.9 U.S. Army Corps of Engineers

Publications and computer programs, too numerous to list, are available from the U.S. Army Corps of Engineer's Publication website. A publication catalog may be obtained by contacting the Hydrologic Engineering Center of the Corp, 609 Second St., Davis, CA 95616. The U. S. Army Corps of Engineers publications website address is: https://www.publications.usace.army.mil/

Topic 808 – Selected Computer Programs

Table 808.1 below presents a software vs. capabilities matrix for hydrologic/hydraulic software packages that have been reviewed and deemed compatible with Departmental procedures. Where Caltrans drainage facilities connect or impact facilities that are owned by others, the affected Local Agency may require the Department to use a specific program that is not listed below. When the use of other computer programs is requested, a comparison with the results using the appropriate program from Table 808.1 should be made. However, when work is performed on projects under Caltrans' jurisdiction, either internally, or by others, if a program not listed in Table 808.1 is used, it should be demonstrated that the computations are based on the same principles that are used in the programs listed in Table 808.1. For information on Local Agency hydraulic computer program requirements, the District Hydraulics Branch should be contacted. It is the responsibility of the user to ensure that the version of the program being used from Table 808.1 is current.

Table 808.1

Summary of Related Computer Programs and Web Applications

	Storm Drains	Hydrology	Water Surface Profiles	Culverts	Roadside /Median Channels	Pavement Drainage	Pond Routing
FHWA Hydraulic Toolbox					x	x	
TR-55		x					
HEC-HMS [(2)]		x					x
HY-8				x			
HEC-RAS [(1)]			x				
FESWMS			x				
WMS		x		x			x
NOAA Atlas 14		x					
USGS StreamStats		x					
SMS/SRH-2D			x	x			x
AutoDesk Civil3D/ Hydraflow	x	x				x	x

NOTES:

[(1)]The data that was used by FEMA to establish water surface elevations (usually HEC-2) must be used to develop a duplicate effective model for FEMA floodplain analysis. For more information contact FEMA or the Local Agency.

[(2)]HEC-1 has been superseded by HEC-HMS by the U.S. Army Corps of Engineers.

Special circumstances may dictate the use of alternative methods/programs. Any such use should be performed under direction and with approval of the District Hydraulics Engineer.

CHAPTER – 820 CROSS DRAINAGE

Topic 821 – General

Index 821.1 – Introduction

Cross drainage involves the conveyance of surface water and stream flow across or from the highway right of way. This is accomplished by providing either a culvert or a bridge to convey the flow from one side of the roadway to the other side or past some other type of flow obstruction. Some culverts and bridges will also have to convey various species of wildlife within known migratory routes and movement corridors of wildlife connectivity areas. These wildlife connectivity areas are identified through the collaboration between Caltrans and California Department of Fish and Wildlife. Culverts and bridge crossings that are not within wildlife routes and corridors, and are outside of wildlife connectivity areas, are not required to be designed for wildlife passage.

In addition to the hydraulic function and potential wildlife passage function, a culvert must carry construction and highway traffic and earth loads. Culvert design, therefore, involves both hydraulic and structural design. This section of the manual is basically concerned with the hydraulic design of culverts. Both the hydraulic and structural designs must be consistent with good engineering practice and economics. An itemized listing of good drainage design objectives and economic factors to be considered are listed in Index 801.4. Information on strength requirements, height of fill tables, and other physical characteristics of alternate culvert shapes and materials may be found in Chapter 850, Physical Standards.

More complete information on hydraulic principles and engineering techniques of culvert design may be found in the FHWA Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts". Key aspects of culvert design and a good overview of the subject are more fully discussed in the AASHTO Highway Drainage Guidelines.

Structures measuring more than 20 feet along the roadway centerline are conventionally classified as bridges, assigned a bridge number, and maintained and inspected by the Division of Structures. However, some structures classified as bridges are designed hydraulically and structurally as culverts. Some examples are certain multi-barreled box culverts and arch culverts. Culverts, as distinguished from bridges, are usually covered with embankment and have structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert.

Bridges are not designed to take advantage of submergence to increase hydraulic capacity even though some are designed to be inundated under flood conditions. For economic and hydraulic efficiency, culverts should be designed to operate with the inlets submerged during flood flows, if conditions permit. At many locations, either a bridge or a culvert will fulfill both the structural and hydraulic requirements of the stream crossing. Structure choice at these locations should be based on construction and maintenance costs, risk of failure, risk of property damage, traffic safety, and environmental and aesthetic considerations.

821.5 Effects of Tide and Storm

Culvert outfalls and bridge openings located where they may be influenced by ocean tides require special attention to adequately describe the 1% Annual Exceedance Probability (AEP).

Detailed statistical analysis and use of unsteady flow models, including two-dimensional models, provide the most accurate approach to describing the combined effects of tidal and meteorological events. Such special studies are likely warranted for major hydraulic structures (See HEC-25, 3rd edition, 2020 - "Highways in the Coastal Environment"), but would typically be too costly and time consuming for lesser facilities. If the risk factors and costs associated with a failure of the drainage facility (such as, bridge or culvert) located in a tidal environment do not support conducting such a detailed analysis, the following guidance can be used to select ocean or bay water levels and flood events to adequately estimate the 1% AEP. However, the effect of climate change or sea-level rise is not included in this analysis. Sea-level rise needs to be evaluated for all coastal facilities using Section 883.2 ("Design High Water, Design Wave Height and Sea-Level Rise") of this manual or any other appropriate method.

The daily maximum ocean water levels vary significantly on a fortnightly basis with the spring-neap cycle, where the highest daily maximum water levels occur during spring tides and the lowest daily maximum water levels occur during neap tides. The annualized probability of the daily maximum ocean water level $\hat{\eta}_T$, with a return period T year, that may exceed a certain elevation can be expressed using a stage-frequency relationship. Such a relationship has been developed using the water level data received from the National Oceanic and Atmospheric Administration (NOAA) tide gauge stations located in the California coast. These gauge stations typically record water levels every six minutes, and those measurements account for all the combined astronomical, meteorological and climatic effects that have influenced the water levels in the coastal regions of California. The NOAA has periodically verified those ocean water levels for multi-decadal periods which are referred to as "tidal epochs." The basis for developing the Annual Exceedance Probability (AEP) for ocean water levels reaching or exceeding a particular elevation in a day is first, finding the ratio of the total number of daily maximums water levels that reach or exceed that elevation over the total number of daily maximum water level measurements in each year and then averaging the result over the years that make up the period of record of that tide gauge. Finally, these processes are repeated for a range of elevations to develop a continuous relationship with the corresponding AEP. Figure 821.1 shows an example of the continuous distribution where the daily maximum ocean water level for outer San Francisco Bay is plotted against the AEP expressed in percentage. This curve has been derived based on NOAA tide gauge station 9414290 for period of record June 30, 1854 to present. AEP for some tidal datums are also shown here. For this location, the annual probability of the daily maximum ocean water level exceeding the Mean High Water (MHW) is 73%. It is to be noted that all tidal datums in this analysis are based on the tidal epoch 1983 to 2001.

821.6 Wildlife Passage Consideration

As stated in Section 158 of the California Streets and Highway Code, wildlife paths and corridors within wildlife connectivity areas should be investigated and assessed by Caltrans in consultation with California Department of Fish and Wildlife (CDFW) and other appropriate agencies.

The first objective for these investigations is to create a Caltrans statewide inventory of wildlife connectivity needs with the goal of reducing wildlife-vehicle collisions and enhancing wildlife connectivity. Some significant or important factors that need to be considered in the development of the inventory are cost of implementing wildlife passage features, ease of acquiring land for ecologic buffer, level of significance of project to wildlife connectivity, and longevity of wildlife passage features in their enhancement of connectivity and public safety.

The second objective of these investigations is to assess barriers within wildlife connectivity areas on a per-project basis based on specific project conditions. According to Section 158, wildlife connectivity assessments will be performed for any projects that is starting the PID phase on or after July 1, 2025, that either adds a traffic lane or that will have a potential for significantly impacting wildlife connectivity for target species in a connectivity area, determined by the criteria developed through a collaborative effort between Caltrans and CDFW.

Based on the wildlife connectivity inventory and the possible wildlife connectivity project assessment, the decision to provide wildlife passage for a project will be determined by the project development team in consultation with the California Department of Fish and Wildlife. If a project meets appropriate criteria, the investigation and assessment will determine whether barriers exist to the movement of target wildlife species. On a per-project basis, wildlife barriers will be assessed, and remediation of identified barriers will be evaluated.

For cross drainage locations that have known wildlife routes and corridors with identified barriers, barrier remediation strategies may require the improvement or replacement of existing culverts and bridges that provide passage of creeks, streams, rivers, and washes. When it is determined that wildlife is using these types of perennial or ephemeral channels for migration and movement, a culvert or bridge will have to convey wildlife in addition to flood flows. This is called wet wildlife crossing and cross drainage structures need to meet hydraulic and hydrologic requirements outlined in Chapter 820, Cross Drainage, as well as the requirements for wildlife passage.

In addition to wet wildlife crossings, wildlife may also use overland corridors, such as floodplains, for their movement and migration, which are referred to as dry wildlife crossings. These types of crossings can incorporate various types of culvert and bridge facilities to provide wildlife passage under or over a roadway, highway, or freeway. As a barrier remediation strategy, new dry wildlife crossings facilities may need to be designed and constructed.

If wildlife passage is required, refer to “Caltrans Wildlife Crossing and Connectivity Design Guidelines” for all wildlife passage design aspects of wet and dry wildlife crossings. This document is located at the Caltrans Division of Design, Office of Hydraulics and Stormwater website.

Topic 822 – Debris Control

822.1 Introduction

Debris, if allowed to accumulate either within a culvert or at its inlet, can adversely affect the hydraulic performance of the facility. Damage to the roadway and to upstream property may result from debris obstructing the flow into the culvert. Coordination with district maintenance forces can help in identifying areas with high debris potential and in setting requirements for debris removal where necessary.

The use of any device that can trap debris must be thoroughly examined prior to its use. In addition to the more common problem of debris accumulation at the culvert entrance, the use of safety end grates or other appurtenances can also lead to debris accumulation within the culvert at the outlet end. Evaluation of this possibility, and appropriate preventive action, must be made if such end treatment is proposed.

822.2 Debris Control Methods

There are two methods of handling debris:

(1) Passing Through Culvert. If economically feasible, culverts should be designed to pass debris. Culverts which pass debris often have a higher construction cost. On the other hand, retaining solids upstream from the entrance by means of a debris control structure often involves substantial maintenance cost and could negatively affect fish passage. An economic comparison which includes evaluation of long term maintenance costs should be made to determine the most reasonable and cost effective method of handling.

(2) Interception. If it is not economical to pass debris, it should be retained upstream from the entrance by means of a debris control structure or the use of a debris basin when the facility is located in the vicinity of alluvial fans.

If drift and debris are retained upstream, a riser or chimney may be required. This is a vertical extension to the culvert which provides relief when the main entrance is plugged. The increased head should not be allowed to develop excessive velocities or cause pressure which might induce leakage in the culvert.

If debris control structures are used, access must be provided for maintenance equipment to reach the site. This can best be handled by coordination and field review with district maintenance staff. Details of a pipe riser with debris rack cage are shown on Standard Plan D93C. See FHWA Hydraulic Engineering Circular No. 9, "Debris-Control Structures" for further information.

The use of an upstream debris basin and downstream concrete lined channels, has often been used by Local Agencies for managing flood flows on alluvial fans in urbanized areas. Experience has shown that this approach is effective, however, the costs of building and maintaining such facilities is high with a potential for sediment inflows greater than anticipated.

825.3 Computer Programs

Numerous calculator and computer programs are available to aid in the design and analysis of highway culverts. The major advantages of these programs over the traditional hand calculation method are:

- Increased accuracy over charts and nomographs.
- Rapid comparison of alternative sizes and inlet configurations.

Familiarity with culvert hydraulics and traditional methods of solution is necessary to provide a solid basis for designers to take advantage of the speed, accuracy, and increased capabilities of hydraulic design computer programs.

The hydraulic design calculator and computer programs available from the FHWA are more fully described in HDS No. 5, "Hydraulic Design of Highway Culverts."

The H-Y8 culvert hydraulics program provides interactive culvert analysis. Given all of the appropriate data, the program will compute the culvert hydraulics for circular, rectangular, elliptical, arch, and user-defined culverts.

The logic of H-Y8 involves calculating the inlet and outlet control headwater elevations for the given flow. The elevations are then compared and the larger of the two is used as the controlling elevation. In cases where the headwater elevation is greater than the top elevation of the roadway embankment, an overtopping analysis is done in which flow is balanced between the culvert discharge and the surcharge over the roadway. In the cases where the culvert is not full for any part of its length, open channel computations are performed. In locations where tailwater is an important factor in the culvert design, HY-8 is limited because it must assume a tailwater (or simple channel calculation) and cannot calculate the tailwater based on the stream flow beyond the culvert.

Culverts can also be modeled in a two-dimensional (2D) domain using the “3D Structure” feature in SMS/SRH-2D. This procedure of calculating the flow through the culvert is superior to that of HY-8, particularly when dealing with culverts with large openings. This modeling and analysis preserves the approach velocity and momentum. Usually, box culverts, circular culverts, or arch culverts on the main channel can be modeled using this approach. The ability to model within the culvert in the 2D domain ensures that the flow’s momentum at the culvert is maintained at the inflow, which is not possible using HY-8. Any oblique flow or any variation of flow across the culvert section initiated upstream can be introduced and carried through the 2D domain within the culvert utilizing SMS/SRH-2D.

825.4 Coefficient of Roughness

Suggested Manning's n values for culvert design are given in Table 852.1.

Whether warped or straight flared wingwalls are used depends on the shape of the approach channel. Straight flared wingwalls are appropriate for well defined channels with steep banks. Warped wingwalls are more suited to shallow trapezoidal approach channels.

Usually it is more economical to transition between the stream section and the culvert by means of straight flared wingwalls or warped wingwalls than to expand the culvert barrel at entrance. For a very wide channel, this transition may be combined with riprap, dikes, or channel lining extending upstream to complete the transition.

(2) *Transitions.* Elaborate transitions and throated openings for culverts may be warranted in special cases. Generally a highly developed entrance is unnecessary if the shape of the culvert fits the approach channel. In wide flat channels where ponding at entrance must be restricted, a wide shallow structure or multiple conduit should be used if drift and debris are not a problem.

Throated or tapered barrels at entrance are more vulnerable to clogging by debris. They are not economical unless they are used for corrective measures; for example, where there is a severe restriction in right of way width and it is necessary to increase the capacity of an existing culvert structure.

For further information refer to HEC-9, "Debris-Control Structures" and HDS 5, "Hydraulic Design of Highway Culverts"

Topic 827 – Outlet Design

827.1 General

The outlet velocity of highway culverts is usually higher than the maximum natural stream velocity. This higher velocity can cause streambed scour and bank erosion for a limited distance downstream from the culvert outlet.

The slope and roughness of the culvert barrel are the principle factors affecting outlet velocity. The shape and size of a culvert seldom have a significant effect on the outlet velocity. When the outlet velocity is believed to be excessive and it cannot be satisfactorily reduced by adjusting the slope or barrel roughness, it may be necessary to use some type of outlet protection or energy dissipator. A method of predicting and analyzing scour conditions is given in Chapter 5 of the FHWA HEC-14 manual.

When dealing with erosive velocities at the outlet, the effect on downstream property should be evaluated.

827.2 Embankment Protection

Improved culvert outlets are designed to restore natural flow conditions downstream. Where erosion is to be expected, corrective measures such as bank protection, vertical flared wingwalls, warped wingwalls, transitions, and energy dissipators may be considered. See Chapter 870, "Channel and Shore Protection-Erosion Control", FHWA Hydraulic Engineering Circulars, No. 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels", No. 15, "Design of Roadway Channels with Flexible Linings", No. 23 “Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance, and "Hydraulic Design of Stilling Basins and Energy Dissipators", Engineering Monograph No. 25 by the U. S.

(4) *Outlet Treatment.* Where excessive erosion at an overside drain outlet is anticipated, a simple energy dissipater should be employed. Preference should be given to inexpensive expedients such as an apron of broken concrete or rock, a short section of pipe placed with its axis vertical with the lowermost 6 inches filled with coarse gravel or rock, or a horizontal tee section which is usually adequate for downdrain discharges.

(5) *Anchorage*. For slopes flatter than 3:1 overside drains do not need to be anchored. For slopes 3:1 or steeper overside drains should be anchored with 6 foot pipe stakes as shown on the Standard Plans to prevent undue strain on the entrance taper or pipe ends. For drains over 150 feet long, and where the slope is steeper than 2:1, cable anchorage should be considered as shown on the Standard Plans. Where the cable would be buried and in contact with soil, a solid galvanized rod should be used the buried portion and a cable, attached to the rod, used for the exposed portion. Beyond the buried portion, a slip joint must be provided when the installation exceeds 60 feet in length. Regard-less of pipe length or steepness of slope, where there is a potential for hillside movement cable anchorage should be considered.

When cable anchorage is used as shown on the Standard Plans, the maximum allowable downdrain lengths shall be 200 feet for a slope of 1.5:1 and 250 feet for a slope of 2:1. For pipe diameters greater than 24 inches, or downdrains to be placed on slopes steeper than 1.5:1, special designs are required. Where there is an abrupt change in direction of flow, such as at the elbow or a tee section downstream of the end of the cable anchorage system, specially designed thrust blocks should be considered.

(6) *Drainage on Benches.* Drainage from benches in cut and fill slopes should be removed at intervals ranging from 300 feet to 500 feet.

(7) *Selection of Types.* Pipe and flume downdrains may consist of either corrugated steel, corrugated aluminum, or any other approved material that meets the minimum design service life required under Chapter 850. Refer to Index 855.2 for additional discussion on limitations of abrasive resistance of aluminum pipe culverts.

Topic 835 – Dikes and Berms

835.1 General

Dikes and berms are to be used only as necessary to confine drainage and protect side slopes susceptible to erosion.

835.2 Earth Berms

Earth Berms should be designed in accordance with HEC-15, Design of Roadside Channels with Flexible Linings, and USACE Engineering Manual (EM) 1110-2-2300, General Design and Construction Considerations for Earth and Rock-Fill Dams, which is available at:

https://www.publications.usace.army.mil/USACE-Publications/Engineer-Manuals/

835.3 Dikes

Details of dikes are shown on Standard Plan A87. See Topic 303 for a detailed discussion on the types and placement considerations for dikes.

Topic 837 – Inlet Design

837.1 General

The basic features of standard storm drain inlets are shown in Figure 837.1. Full details appear on Standard Plan D72 through D75, D98-A and D98-B. The variety of standard designs available is considered sufficient to any drainage situation; hence, the use of nonstandard inlets should be rare.

837.2 Inlet Types

From an operating standpoint, there are five main groups of inlets; these are:

(1) *Curb-Opening.* Curb opening inlets have an opening parallel to the direction of flow in the gutter. This inlet group is adapted to curb and gutter installations. The curb opening is most effective with flows carrying floating debris. As the gutter grade steepens, their interception capacity decreases. Hence, they are commonly used on grades flatter than 3 percent.

When curb opening inlets are used on urban highways other than fenced freeways, a 3/4 inch plain round protection bar is placed horizontally across any curb or wall opening whose height is 7 inches or more. The unsupported length of bar should not exceed 7 feet. Use of the protection bar on streets or roads under other jurisdiction is to be governed by the desires of the responsible authorities.

The Type OS and OL inlets are only used with Type A or B curbs. A checkered steel plate cover is provided for maintenance access.

The Type OS inlet has a curb opening 3.5 feet long. Since a fast flow tends to overshoot such a short opening, it should be used with caution on grades above 3 percent.

The Type OL inlet is a high capacity unit in which the length of curb opening ranges from 7 feet to 21 feet.

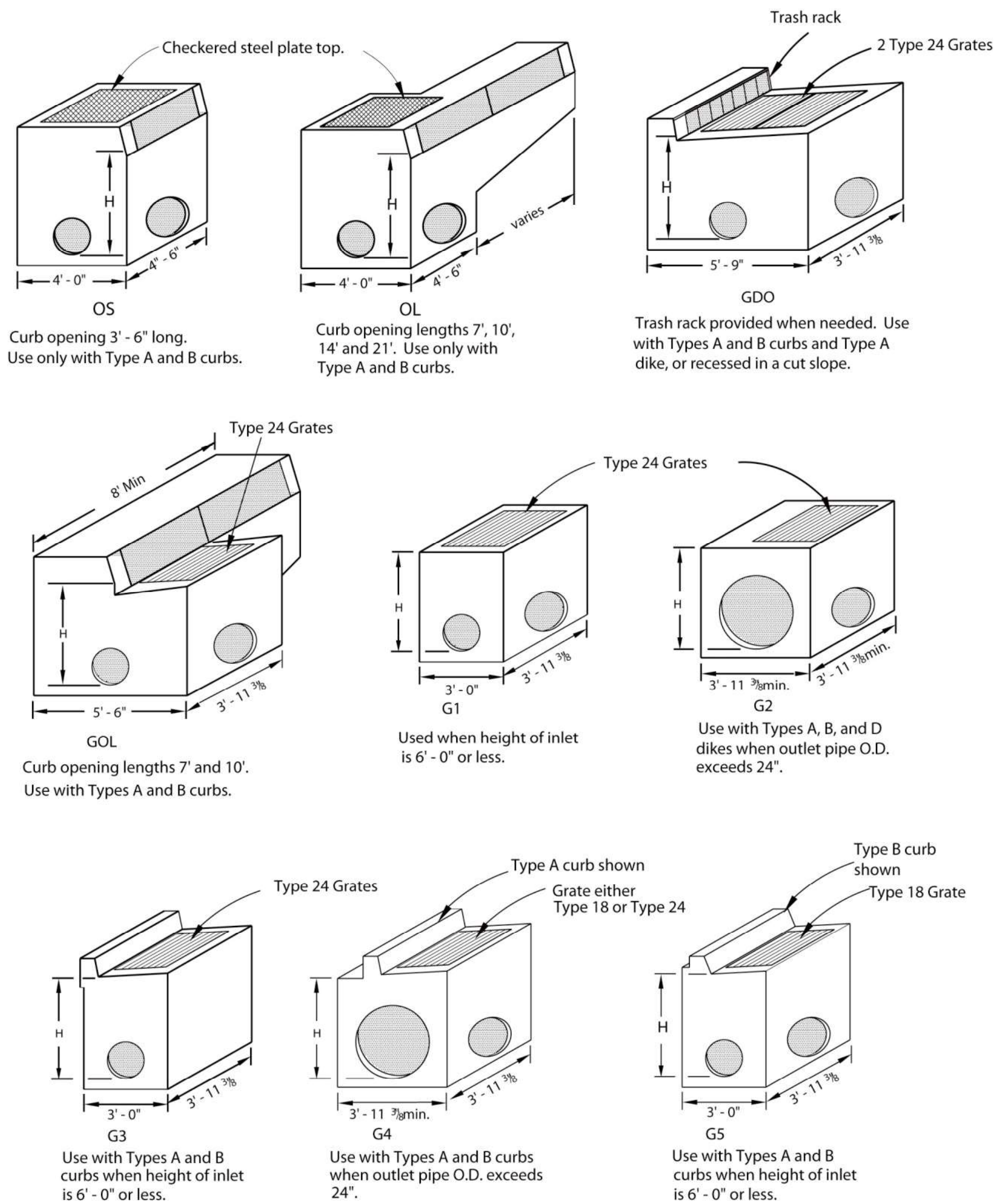
(2) *Grate.* Grate inlets provide a grate opening in the gutter or waterway. As a class, grate inlets perform satisfactorily over a wide range of gutter grades. Their main disadvantage is that they are easily clogged by floating trash and should not be used without a curb opening where total interception of flow is required. They merit preference over the curb opening type on grades of 3 percent or more. Gutter depressions, discussed under Index 837.5, increase the capacity of grate inlets. Grate inlets may also be used at locations where a gutter depression is not desirable. See the Standard Plans for grate details.

Locate grate inlets away from areas where bicycles or pedestrians are anticipated whenever possible. Grate designs that are allowed where bicycle and pedestrian traffic occurs have smaller openings and are more easily clogged by trash and debris and are less efficient at intercepting flow. Additional measures may be necessary to mitigate the increased potential for clogging.

The grate types depicted on Standard Plan D77B must be used if bicycle traffic can be expected. Many highways do not prohibit bicycle traffic, but have inlets where bicycle traffic would not be expected to occur (e.g., freeway median). In such instances, the designer may consider use of grates from Standard Plan D77A. The table of final pay weights on Standard Plan D77B indicates the acceptable grate types to be used for each listed type of inlet.

Figure 837.1

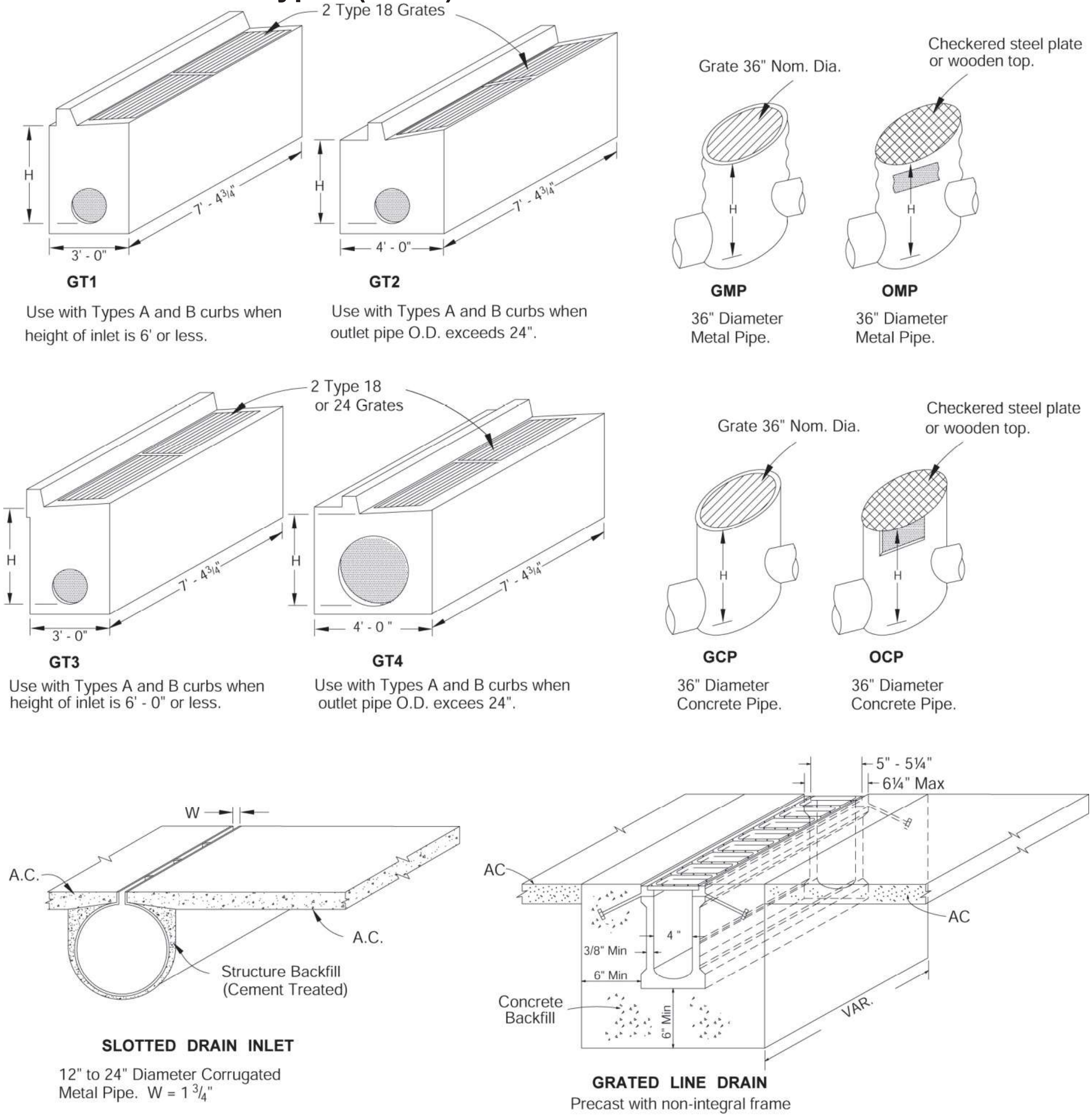
Storm Drain Inlet Types



NOTES:
1. All dimensions are outside dimensions based on 6" wall thickness.
2. For full details on uses according to type, see Index 837.2.
3. H = height of inlet.
4. See Standard Plans for Details.
5. Grates shown are not bicycle proof nor ADA compliant.

Figure 837.1

Storm Drain Inlet Types (Cont.)



NOTES:
1. All dimensions are outside dimensions based on 6" wall thickness.
2. For full details on uses according to type, see Index 837.2.
3. H = height of inlet.
4. See Standard Plans for Details.

If grate inlets must be placed within a pedestrian path of travel, the grate must be compliant with the Americans with Disabilities Act (ADA) regulations which limit the maximum opening in the direction of pedestrian travel to no more than 0.5 inch. Presently, the only standard grating which meets such restrictive spacing criterion is the slotted corrugated steel pipe with heel guard, as shown in the Standard Plans. Because small openings have an increased potential for clogging, a minimum clogging factor of 50 percent should be assumed; however, that factor should be increased in areas prone to significant debris. Other options which may be considered are grated line drains with specialty grates (see the Standard Plans for grated line drain details, and refer to manufacturers catalogs for special application grates) or specially designed grates for standard inlets. The use of specially designed grates is a nonstandard design that must be approved by the Office of Hydraulics and Stormwater Design prior to submittal of PS&E.

(3) *Combination.* Combination inlets provide both a curb opening and a grate. These are high capacity inlets which make use of the advantages offered by both kinds of openings.

(a) Type GO and GDO. These types of inlets have a curb opening directly opposite the grate. The GDO inlet has two grates placed side by side and is designed for intercepting a wide flow. A typical use of these inlets would be in a sag location either in a curb and gutter installation or within a shoulder fringed by a dike. When used as the surface inlet for a pumping installation, the trash rack shownontheStandardPlanD74Bisprovided.

(b) Type GOL. This is called a sweeper inlet because the curb opening precedes the grate. It is particularly useful as a trash interceptor during the initial phases of a storm. When used in a grade sag, the sweeper inlet can be modified by providing a curb opening on both sides of the grate.

(4) *Pipe.* Pipe drop inlets are made of a commercial pipe section of concrete or corrugated metal. As a class, they develop a high capacity and are generally the most economical type. This type of inlet is intended for uses outside the roadbed at locations that will not be subjected to normal highway wheel loads.

Two kinds of inlets are provided; a wall opening and a grate top. The wall opening inlet should only be used at protected locations where it is unlikely to be hit by an out of control vehicle.

(a) Wall Opening Intake. This opening is placed normal to the direction of surface flow. It develops a high capacity unaffected by the grade of the approach waterway. The inlet capacity is increased by depressing the opening; also by providing additional openings oriented to intercept flows from different directions. When used as the surface intake to a pumping installation, a trash rack across the opening is required. See Standard Plans for pipe inlet details. Because this type of inlet projects above grade, its use should be avoided in areas subject to traffic leaving the roadway.

(b) Grate Intake. The grate intake intercepts water from any direction. For maximum efficiency, however, the grate bars must be in the direction of greatest surface flow. Being round, it is most effective for flows that are deepest at the center, as in a valley median.

(5) *Slotted Drains.* This type of inlet is made of corrugated metal or polyethylene pipe with a continuous slot on top. This type of inlet can be used in flush, all paved medians with superelevated sections to prevent sheet flow from crossing the centerline of the highway. Short sections of slotted drain may be used as an alternate solution to a grate catch basin in the median or edge of shoulder.

(a) Drop inlets or other type of cleanout should be provided at intervals of about 100 feet.

(6) *Grated Line Drains.* This type of inlet is made of monolithic polymer concrete with a ductile iron frame and grate on top. This type of inlet can be used as an alternative at the locations

838.2 Design Criteria

To adequately estimate design storm discharges for a storm drain system in urban areas involving street flooding it may be necessary to route flows by using hydrograph methods. Hydrographs are discussed under Index 816.5 and further information on hydrograph methods may be found in Chapters 8 and 9 of HDS No.2, Highway Hydrology.

838.3 Hydraulic Design

Closed conduits should be designed for the full flow condition. They may be allowed to operate under pressure, provided the hydraulic gradient is 0.75 foot or more below the intake lip of any inlet that may be affected. The energy gradient should not rise above the lip of the intake. Allowances should be made for energy losses at bends, junctions and transitions.

To determine the lowest outlet elevation for drainage systems which discharge into leveed channels or bodies of water affected by tides, consideration should be given to the possibilities of backwater. The effect of storm surges (e.g., winds and floods) should be considered in addition to the predicted tide elevation.

Normally, special studies will be required to determine the minimum discharge elevation consistent with the design discharge of the facility.

838.4 Standards

(1) *Location and Alignment.* Longitudinal storm drains are not to be placed under the traveled way of highways. Depending upon local agency criteria, storm drains under the traveled way of other streets and roads may be acceptable. A manhole or specially designed junction structure is usually provided at changes in direction or grade and at locations where two or more storm drains are joined. Refer to Index 838.5 for further discussion on manholes and junction structures.

(2) *Pipe Diameter.* The minimum pipe diameter to be used is given in Table 838.4.

(3) *Slope.* The minimum longitudinal slope should be such that when flowing half full, a self cleaning velocity of 3 feet per second is attained.

(4) *Physical Properties.* In general, the considerations which govern the selection of culvert type apply to storm drain conduits. Alternative types of materials, overfill tables and other physical factors to be considered in selecting storm drain conduit are discussed under Chapter 850.

(5) *Storage.* In developing the most economical installation, the designer should not overlook economies obtainable through the use of pipeline storage and, within allowable limits, the ponding of water in gutters, medians and interchange areas. Inlet capacity and spacing largely control surface storage in gutters and medians; inlet capacity governs in sump areas.

Table 838.4

Minimum Pipe Diameter for Storm Drain Systems

Type of Drain	Minimum Diameter (in)
Trunk Drain	18
Trunk Laterals	15[1]
Inlet Laterals	15[1]

NOTE:

[1]18 minimum if wholly or partly under the roadbed.

Specific subjects for special consideration are:

- Bedding and Backfill. Bedding and backfill consideration are discussed under Index 829.2. Maximum height of cover tables are included in Chapter 850 and minimum thickness of cover is given in Table 856.5.
- Roughness Factor. The roughness factor, Manning's n value, generally assumes greater importance for storm drain design than it does for culverts. Suggested Manning's n values for various types of pipe materials are given in Table 852.1.

(6) *Floating Trash.* Except at pumping installations, every effort should be made to carry all floating trash through the storm drain system. Curb and wall opening inlets are well suited for this purpose. In special cases where it is necessary to exclude trash, as in pumping installations, a standard trash rack must be provided across all curb and wall openings of tributary inlets. See the Standard Plans for details.

(7) *Median Flow.* In estimating the quantity of flow in the median, consideration should be given to the effects of trash, weeds, and plantings.

838.5 Appurtenant Structures

(1) *Manholes.*

(a) General Notes. The purpose of a manhole is to provide access to a storm drain for inspection and maintenance. Manholes are usually constructed out of cast in place concrete, pre-cast concrete, or corrugated metal pipe. They are usually circular and approximately three or four feet in diameter to facilitate the movement of maintenance personnel.

There is no Caltrans Standard Plan for manholes. Relocation and reconstruction of existing storm drain facilities, owned by a city or county agency, is often necessary. Generally the local agency has adopted manhole design standard for use on their facilities. Use of the manhole design preferred by the responsible authority or owner is appropriate.

Commercial precast manhole shafts are effective and usually more economical than cast in place shafts. Brick or block may also be used, but only upon request and justification from the local agency or owner.

(b) Location. Following are common locations for manholes:

- Where two or more drains join,
- At locations and spacing which facilitate maintenance,
- Where the drain changes in size,
- At sharp curves or angle points in excess of 10 degrees,
- Points where an abrupt flattening of the grade occurs, and
- On the smaller drains, at the downstream end of a sharp curve.

Manholes are not required if the conduit is large enough to accommodate a man, unless spacing criteria govern. Manholes should not be placed within the traveled way. Exceptions are frontage roads and city streets, but intersection locations should be avoided.

(c) Spacing. In general, the larger the storm drain, the greater the manhole spacing. For pipe diameter of 48 inches or more, or other shapes of equal cross sectional area, the manhole spacing ranges from 700 feet to 1200 feet. For diameters of less than 48 inches, the spacing may vary from 300 feet to 700 feet. In the case of small drains where self-cleaning velocities are unobtainable, the 300 feet spacing should be used. With self-cleaning velocities and alignments without sharp curves, the distance between manholes should be in the upper range of the above limits.

(d) Access Shaft. For drains less than 48 inches in diameter, the access shaft is to be centered over the drain. When the drain diameter exceeds the shaft diameter, the shaft should be offset and made tangent to one side of the pipe for better location of the manhole steps. For drains 48 inches or more in diameter, where laterals enter from both sides of the manhole, the offset should be toward the side of the smaller lateral. See Standard Plan D93A for riser connection details.

(e) Arrangement of Laterals. To avoid unnecessary head losses, the flow from laterals which discharge opposite each other should converge at an angle in the direction of flow. If conservation of head is critical, a training wall should be provided.

(2) Junction Structures. A junction structure is an underground chamber used to join two or more conduits, but does not provide direct access from the surface. It is designed to prevent turbulence in the flow by providing a smooth transition. This type of structure is usually needed only where the trunk drain is 42 inches or more in diameter. A standard detail sheet of a junction structure is available for pipes ranging from 42 inches to 84 inches in diameter and can be found at Caltrans Engineering Services (DES) website. The XS sheet reference is XS 4-26. Where required by spacing criteria, a manhole should be used.

(3) Flap Drainage gates. When necessary, backflow protection should be provided in the form of flap drainage gates. These gates offer negligible resistance to the release of water from the system and their effect upon the hydraulics of the system may be neglected.

If the outlet is subject to floating debris, a shelter should be provided to prevent the debris from clogging the flap drainage gate. Where the failure of a flap drainage gate to close would cause serious damage, a manually controlled gate in series should be considered for emergencies.

- *Cleanouts*. Terminal and intermediate risers may be placed for the convenience of the maintenance forces cleaning the system. When practical, a terminal riser should be placed at the upper end of an underdrain. Intermediate cleanout risers may be placed at intervals of 500 feet and at sharp angle points greater than 10 degrees.

  The diameter of risers should be the same as the pipe underdrain. Details of underdrain risers are shown on Standard Plan D102.

- *Grade*. If possible, pipe underdrains should be placed on grades steeper than 0.5 percent. Minimum grades of 0.2 percent for laterals and 0.25 percent for mains are acceptable.

- *Depth and Spacing*. The depth of the underdrain depends on the permeability of the soil, the elevation of the water table, and the amount of drawdown needed to ensure stability. Whenever practicable, an underdrain pipe should be set in the impervious zone below the aquifer. Additionally, consideration should be given to the depth and proximity of storm drains. Typically, the underdrain should be placed at a depth sufficient to keep the storm drain above the groundwater table.

  Table 842.4 gives suggested depths and spacing of underdrains according to soil types. It is only a guide and should not be considered a substitute for field observations or local experience.

842.5 Types of Underdrain Pipe

The aim of any underdrain installation is long term effectiveness. This aim is associated with filtering ability, durability, strength, and cost of conduit, mainly in that order. In choosing between pipes of different types, the key considerations are filtering ability and durability. Pipe cost assumes secondary importance because it is a minor part of the underdrain investment.

Pipes for underdrains are perforated and may be made of steel, aluminum, polyvinyl chloride (PVC) or polyethylene, all with corrugated profiles, or smooth wall PVC. All of the listed types are acceptable for either shallow or deep burial situations. Where plastic pipe underdrains are proposed and burial depths would exceed 30 feet, the Underground Structures Unit in the Division of Engineering Services should be contacted for approval.

842.6 Design Service Life

Refer to Chapter 850 for further discussion and criteria relative to design service life of pipe materials used in underdrain installations.

Experience with underdrains has shown that they are not subject to corrosion in an environment that lacks an adequate supply of air and oxygen entrained in the water. Subsurface waters that may be inclined to be corrosive chemically do not tend to become so as long as they are not exposed to oxygen. However, subsurface water may become corrosive after it has surfaced and been exposed to oxygen. Furthermore, there is evidence that indicates there is little oxygen available in long lengths of the small diameter pipe normally used in a subsurface drainage system.

Although tests may indicate that corrosive salts are present in the soil solution, corrosion will not take place without the presence of oxygen. Therefore, when it is anticipated that the

Table 842.4

Suggested Depth and Spacing of Pipe Underdrains for Various Soil Types

Soil Class	Soil Composition			Drain Spacing (ft)			
	Percent Sand	Percent Silt	Percent Clay	3 feet Deep	4 feet Deep	5 feet Deep	6 feet Deep
Clean Sand	80-100	0-20	0-20	110-150	150-200	- -	- -
Sandy Loam	50-80	0-50	0-20	50-100	100-150	- -	- -
Loam	30-50	30-50	0-20	30-60	40-80	50-100	60-120
Clay Loam	20-50	20-50	20-30	20-40	25-50	30-60	40-80
Sandy Clay	50-70	0-20	30-50	15-30	20-40	25-50	3-60
Silty Clay*	0-20	50-70	30-50	10-25	15-30	20 40	25-50
Clay*	0-50	0-50	30-100	15 (max)	20 (max)	25 (max)	40 (max)

*Drainage blankets or stabilization trenches should be considered.

underdrain will be placed to intercept groundwater under the above conditions, it will not be necessary to allow for metal pipe corrosion.

When the above conditions do not prevail, the design service life of metal pipe is determined from pH and resistivity tests covered in California Test 643. This information is shown in the Geotechnical Design Report. The design service life of steel pipe may be increased by a applying a coating as indicated in Table 855.2C.

The guide values contained in the tables mentioned above may be modified where field observation of existing installations dictates.

842.7 Pipe Selection

In cases where more than one material meets the foregoing requirements, alternatives should be specified on the basis of optional selection by the contractor. The selection of a single type of underdrain may be appropriate due to other related factors. This selection should be supported by complete analysis of factors and documentation placed on file in the District.

the use of precast box culverts is applicable, the project plans should include them as an alternative to cast-in-place construction. Because the standard measurement and payment clauses for precast RCB's differ from cast-in-place construction, precast units must be identified as an alternative and the special provision must be appropriately modified.

The standard plan sheets for precast boxes show details which require them to be layed out with joints perpendicular to the centerline of the box. This is a consideration for the design engineer in situations which require stage construction and when the culvert is to be aligned on a high skew. This situation will require either a longer culvert than otherwise may have been needed, or a special design allowing for skewed joints. Prior to selecting the latter option DES - Structures Design should be consulted.

(2) *Concrete Arch Culverts.* Technical questions regarding concrete arch culverts should be directed to the Underground Structures Branch of DES - Structures Design.

(3) *Three-Sided Concrete Box Culverts* Design details for cast-in-place (CIP) construction three-sided bottomless concrete box culverts in 2-foot span increments from 12 feet to < 20 feet, inclusive, with strength classifications shown for 10 feet and 20 feet overfills are available upon request from DES - Structures Design. CIP Bottomless Culvert XS-sheets 17-050-1, 2, 3, 4 and 5 may be obtained electronically. Precast three-sided box culverts are an acceptable alternative to CIP designs, where contractors may submit such designs for approval. Both precast and CIP designs must be placed on a foundation designed specifically for the project site.

(4) *Corrosion, Abrasion, and Invert Protection.* Refer to Index 855.2 Abrasion, and Index 855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates for corrosion, abrasion and invert protection of concrete box and arch culverts.

852.3 Corrugated Steel Pipe, Steel Spiral Rib Pipe and Pipe Arches

Corrugated steel pipe, steel spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. For larger diameters, arch spans or special shapes, see Index 852.5. Corrugated steel pipe and pipe arches are available in various corrugation profiles with helical and annular corrugations. Corrugated steel spiral rib pipe is available in several helical corrugation patterns.

(1) *Hydraulics.* Annular and helical corrugated steel pipe configurations are applicable in the situations where velocity reduction is important or if a culvert is being designed with an inlet control condition. Spiral rib pipe, on the other hand, may be more appropriate for use in stormdrain situations or if a culvert is being designed with an outlet control condition. Spiral rib pipe has a lower roughness coefficient (Manning's "n") than other corrugated metal pipe profiles.

(2) *Durability.* The anticipated maintenance-free service life of corrugated steel pipe, steel spiral rib pipe and pipe arch installations is primarily a function of the corrosivity and abrasiveness of the environment into which the pipe is placed. Corrosion potential must be determined from the pH and minimum resistivity tests covered in California Test 643. Abrasive potential must be estimated from bed material that is present and anticipated flow velocities. Refer to Index 855.1 for a discussion of maintenance-free service life and Index 855.2 Abrasion, and Index 855.3 Corrosion.

The following measures are commonly used to prolong the maintenance-free service life of steel culverts:

(a) Galvanizing. Under most conditions plain galvanizing of steel pipe is all that is needed; however, the presence of corrosive or abrasive elements may require additional protection.

- Protective Coatings – The necessity for any coating should be determined considering hydraulic conditions, local experience, possible environmental impacts, and long-term economy. Approved protective coatings are polymeric sheet, which can be applied to the inside and/or outside of the pipe; and polyethylene for composite steel spiral ribbed pipe which is a steel spiral ribbed pipe externally pre-coated with a polymeric sheet, and internally polyethylene lined. All of these protective coatings are typically shop-applied prior to delivery to the construction site. Polymeric sheet coating provides much improved corrosion resistance and can be considered to typically allow achievement of a 50-year maintenance-free service life without need to increase thickness of the steel pipe. To ensure that a damaged coating does not lead to premature catastrophic failure, the base steel thickness for pipes that are to be coated with a polymeric sheet must be able to provide a minimum 10-year service life prior to application of the polymeric material.

  Bituminous coatings are no longer used as protective coatings for metal pipes. Citing Section 5650 of the Fish and Game Code, the Department of Fish and Game (DFG) has restricted the use of bituminous coatings on the interior of pipes if they are to be placed in streams that flow continuously or for an extended period (more than 1 to 2 days) after a rainfall event. Their concern is that abraded particles of asphalt could enter the stream and degrade the fish habitat. Where abrasion is unlikely, DFG concerns should be minimal. DFG has indicated that they have no concerns regarding interior application of polymeric sheet coatings, even under abrasive conditions.

  Where the materials report indicates that soil side corrosion is expected, an exterior application of polymeric sheet, as provided in the Standard Specifications, combined with galvanizing of steel, is usually effective in forestalling accelerated corrosion on the backfill side of the pipe. Where soil side corrosion is the only, or primary, factor leading to deterioration, a polymeric sheet coating is typically expected to provide up to 50 years of service life to an uncoated pipe. For locations where water side corrosion and/or abrasion is of concern, protective coatings, or protective coatings with pavings, or protective coatings with linings, in combination with galvanizing will add to the culvert service life to a variable degree, depending upon site conditions and type of coating selected. Refer to Index 855.2 Abrasion, and Index 855.3 Corrosion. If hydraulic conditions at the culvert site require a lining on the inside of the pipe or a coating different than that indicated in the Standard Specifications, then the different requirements must be described in the Special Provisions.

- Extra Metal Thickness. – Added service life can be achieved by adding metal thickness. However, this should only be considered after protective coatings and pavings have been considered. Since 0.052 inch thick steel culverts is the minimum steel pipe Caltrans allows, it must be limited to locations that are nonabrasive.

See Table 855.2C for estimating the added service life that can be achieved by coatings and invert paving of steel pipes based upon abrasion resistance characteristics.

(b) Aluminized Steel (Type 2). Evaluations of aluminized steel (type 2) pipe in place for over 40 years have provided data that substantiate a design service life with respect to corrosion resistance equivalent to aluminum pipe. Therefore, for pH values between 5.5 and 8.5, and minimum resistivity values in excess of 1500 ohm-cm, 0.064 inch aluminized steel (type 2) is considered to provide a 50 year design service life. Where abrasion is of concern, aluminized steel (type 2) is considered to be roughly equivalent to galvanized steel. A concrete invert may also be considered where abrasion is of concern.

For pH ranges outside the 5.5 and 8.5 limits or minimum resistivity values below 1500 ohm-cm, aluminized steel (type 2) should not be used. In no case should the thickness of aluminized steel (type 2) be less than the minimum structural requirements for a given diameter of galvanized steel. Refer to Index 855.2 Abrasion, and Index 855.3 Corrosion.

The AltPipe Computer Program is also available to help designers estimate service life for various corrosive/abrasive conditions. Visit the Division of Design, Office of Hydraulics and Stormwater website to access the AltPipe Computer Program.

(3) *Strength Requirements*. The strength requirements for corrugated steel pipes and pipe arches, fabricated under acceptable methods contained in the Standard Specifications, are given in Tables 856.3A, B, C, & D. For steel spiral rib pipe see Tables 856.3E, F & G.

(a)Design Standards.

- Corrugation Profiles – Corrugated steel pipe and pipe arches are available in 2⅔" x ½", 3" x 1", and 5" x 1" profiles with helical corrugations, and 2⅔" x ½" profiles with annular corrugations. Corrugated steel spiral rib pipe is available in a ¾" x ¾" x 7½" or ¾" x 1" x 11½" helical corrugation pattern. For systems requiring large diameter and/or deeper fill capacity a ¾" x 1" x 8½" helical corrugation pattern is available. Composite steel spiral rib pipe is available in a ¾" x ¾" x 7½" helical ribbed profile.

  Metal Thickness - Corrugated steel pipe and pipe arches are available in the thickness as indicated on Tables 856.3A, B, C & D. Corrugated steel spiral rib pipe is available in the thickness as indicated on Tables 856.3E, F & G. Where a maximum overfill is not listed on these tables, the pipe or arch size is not normally available in that thickness. All pipe sections provided in Table 856.3 meet handling and installation flexibility requirements of AASHTO LRFD. Composite steel spiral rib pipe is available in the thickness as indicated on Table 856.3G.

- Height of Fill – The allowable overfill heights for corrugated steel and corrugated steel spiral rib pipe and pipe arches for the various diameters or arch sizes and metal thickness are shown on Tables 856.3A, B, C, & D. For corrugated steel spiral rib pipe, overfill heights are shown on Tables 856.3E, F & G. Table 856.3G gives the allowable overfill height for composite steel spiral rib pipe.

(4) *Shapes.* Corrugated steel pipe, steel spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. For larger diameters, arch spans or special shapes, see Index 852.5.

(5) *Invert Protection.* Refer to Index 855.2 Abrasion. Invert protection should be considered for corrugated steel culverts exposed to excessive wear from abrasive flows or corrosive water. Severe abrasion usually occurs when the flow velocity exceeds 12 feet per second to 15 feet per second and contains an abrasive bedload of sufficient volume.

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. DIB 83 can be found at the Division of Design, Office of Hydraulics and Stormwater website.

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. DIB 83 can be found at the Division of Design, Office of Hydraulics and Stormwater website.

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 83 at the Division of Design, Office of Hydraulics and Stormwater website.

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, available on the Division of Design, Office of Hydraulics and Stormwater website.

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 Special Provision 71-3.04 includes specifications for preparing the surface of the culvert invert, installing bar reinforcement and anchorage devices, and paving the invert with concrete, shotcrete or authorized cementitious material. For most non-abrasive sites, concrete may comply with the requirements for minor concrete or shotcrete. See index 110.12 Tunnel Safety Orders. Generally, this method is feasible for pipes 48 inches in diameter and larger. If abrasion is present, see Table 855.2F for minimum

material thickness of concrete or authorized material. Concrete should have a minimum compressive strength of 6,000 psi at 28 days and the aggregate source should be harder material than the streambed load and have a high durability index (consult with District Materials Branch for sampling and recommendation). The maximum grading specified (1.5 inch) for coarse aggregate may need to be modified if the concrete must be pumped. The abrasion resistance of cementitious materials is affected by both its compressive strength and hardness of the aggregate. There is a correlation between decreasing the water/cement ratio, increasing compressive strength and increasing abrasion resistance. Therefore, where abrasion is a significant factor, the lowest practicable water/cement ratios and the hardest available aggregates should be used.

Paving thickness will range from 2 inches to 13 inches depending on abrasiveness of site based on Table 855.2A, and paving limits typically vary from 90 to 120 degrees for the internal angle. See Index 855.2 and Table 855.2F. Note that in Table 855.2F cementitious concrete is not recommended for extremely abrasive conditions (Level 6 in Table 855.2A). For extremely abrasive conditions alternative materials are recommended such as abrasion resistant concrete (calcium aluminate), steel plate or adding RSP. Calcium aluminate abrasion resistant concrete or mortar may be selected from the Authorized Materials list for concrete invert paving. If hydraulically feasible, a flattened invert design may be warranted.

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 Hydraulics and Stormwater 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, available on the Division of Design, Office of Hydraulics and Stormwater website.

(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 Hydraulics and Stormwater Design within the Division of Design and the Underground Structures unit of Structures Design within the Division of Engineering Services (DES) is advised.

In case of conflict in the design service life requirements between the above controls, the highest design service life is required except for those cases of interim alignment with more than 10 feet of cover. For temporary construction, a lesser design service life than that shown above is acceptable.

Where the above indicates a minimum design service life of 25 years, 50 years may be used. For example an anticipated change in traffic conditions or when the highway is considered to be on permanent alignment may warrant the higher design service life.

855.2 Abrasion

All types of pipe material are subject to abrasion and can experience structural failure around the pipe invert if not adequately protected. Abrasion is the wearing away of pipe material by water carrying sands, gravels and rocks (bed load) and is dependent upon size, shape, hardness and volume of bed load in conjunction with volume, velocity, duration and frequency of stream flow in the culvert. For example, at independent sites with a similar velocity range, bedloads consisting of small and round particles will have a lower abrasion potential than those with large and angular particles such as shattered or crushed rocks. Given different sites with similar flow velocities and particle size, studies have shown the angularity and/or volume of the material may have a significant impact to the abrasion potential of the site. Likewise, two sites with similar site characteristics, but different hydrologic characteristics, i.e., volume, duration and frequency of stream flow in the culvert, will probably also have different abrasion levels.

In Table 855.2A six abrasion levels have been defined to assist the designer in quantifying the abrasion potential of a site. The designer is encouraged to use the guidelines provided in Table 855.2A in conjunction with Table 855.2B “Bed Materials Moved by Various Flow Depths and Velocities” and the abrasion history of a site (if available) to achieve the required service life for a pipe, coating or invert lining material. Sampling of the streambed materials generally is not necessary, but visual examination and documentation of the size, shape and volume of abrasive materials in the streambed and estimating the average stream slope will provide the designer data needed to determine the expected level of abrasion. Where an existing culvert is in place, the condition of the invert and estimated combined wear rate due to abrasion and corrosion based on remaining pipe thickness measurements or if it is known approximately when first perforation occurred (steel pipe only), should always be used first. Figure 855.3B should be used to estimate the expected loss due to corrosion for steel pipe.

The descriptions of abrasion levels in Table 855.2A are intended to serve as general guidance only, and not all of the criteria listed for a particular abrasion level need to be present to justify defining a site at that level. For example, the use of one of the three lower abrasion levels in lieu of one of the upper three abrasion levels is encouraged where there are minor bedload volumes, regardless of the gradation. See Figure 855.1.

Table 855.2C constitutes a guide for estimating the added service life that can be achieved by coatings and invert paving of steel pipes based upon abrasion resistance characteristics. However, the table does not quantify added service life of coatings and paving of steel pipe based upon corrosion protection. In heavily abrasive situations, concrete inverts or other lining alternatives outlined in Table 855.2A should be considered. The guide values for years of added service life should be modified where field observations of existing installations show that other values are more accurate. The designer should be aware of the following limitations when using Table 855.2C:

- Channel Materials: If there is no existing culvert, it may be assumed that the channel is potentially abrasive to culvert if sand and/or rocks are present. Presence of silt, clay or heavy vegetation may indicate a non-abrasive flow.
- Flow velocities: The velocities indicated in the table should be compared to those generated by the 2-5 year return frequency flood.
- The abrasion levels represent all six abrasion levels presented in Table 855.2A however, levels 2 and 3 have been combined.

Figure 855.1

Minor Bedload Volume

Large, round bedload (top) and RCP with minimal wear and minor bedload volume with moderate to high velocity.

Table 855.2D constitutes a guide for anticipated wear (in mils/year) to metal pipe by abrasive channel materials. No additional abrasion wear is anticipated for steel for the lower three abrasion levels defined in Table 855.2A, because it is assumed that there is some degree of abrasion incorporated within California Test 643 and Figure 855.3B. Figure 855.3B, “Chart for

Estimating Years to Perforation of Steel Culverts," is part of a Standard California Department of Transportation Test Method derived from highway culvert investigations. This chart alone is not used for determining service life because it does not consider the effects of abrasion or overfill; it is for estimating the years to the first corrosion perforation of the wall or invert of the CSP. Additional gauge thickness or invert protection may be needed if the thickness for structural requirements (i.e., for overfill) is inadequate for abrasion potential.

Table 855.2E indicates relative abrasion resistance properties of pipe and lining materials and summarizes the findings from "Evaluations of Abrasion Resistance of Pipe and Pipe Lining Materials Final Report FHWA /CA/TL-CA01-0173 (2007)". This report may be viewed by visiting the Repository & Open Science Access Portal (ROSAP) website. 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.

Under similar conditions, aluminum culverts will abrade between one and a half to three times faster than steel culverts. Therefore, aluminum culverts are not recommended where abrasive materials are present, and where flow velocities would encourage abrasion to occur. Culvert flow velocities that frequently exceed 5 feet per second where abrasive materials are present should be carefully evaluated prior to selecting aluminum as an allowable alternate. In a corrosive environment, Aluminum may display less abrasive wear than steel depending on the volume, velocity, size, shape, hardness and rock impact energy of the bed load. However, if it is deemed necessary to place aluminum pipe in abrasion levels 4 through 6 in Table 855.2C, contact Headquarters Office of Hydraulics and Stormwater Design for assistance.

Aluminized Steel (Type 2) can be considered equivalent to galvanized steel for abrasion resistance and therefore does not have the same limitations as aluminum in abrasive environments.

Concrete pipes typically counter abrasion through increased minimum thickness over the steel reinforcement, i.e., by adding additional sacrificial material. See Table 855.2F. However, there are significantly fewer limitations involved in increasing the invert thickness of RCB in the field verses increasing minimum thickness over the steel reinforcement of RCP in the plant. Therefore, RCP is typically not recommended in abrasive flows greater than 10 feet per second but may be considered for higher velocities if the bedload is insignificant (e.g. storm drain systems and most.

Table 855.2C

Guide for Anticipated Service Life Added to Steel Pipe by Abrasive Resistant Protective Coating[2]

Flow Velocity (ft/s)	Channel Materials	Paved Invert (yrs.)	Polymeric Sheet Coating (yrs.)	Polyethylene (CSSRP) (yrs.)
	Non-Abrasive	15	*	*
≥ 1 – ≤ 8 [1]	Abrasive	15-2	30-5	*
> 8 – ≤ 12	Abrasive	2-0	5-0	70-35
> 12 – ≤ 15	Abrasive	**	**	35-8***
> 12 – ≤ 20	Abrasive & heavy bedloads	****	****	****

* Provides adequate abrasion resistance to meet or exceed a 50-year design service life.
** Abrasive resistant protective coatings not recommended, increase steel thickness to 10 gage.
*** Not recommended above 14 fps flow velocity.
**** Contact District Hydraulics Branch. See Table 855.2F.

Notes:
[1] Where there are increased velocities with minor bedload volumes, much higher velocities may be applicable.
[2] Range of additional service life commensurate with flow velocity range.

Table 855.4B

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 Hydraulics and Stormwater 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. Refer to Cal Fire's "Fire Hazard Severity Zone Maps" website from the Office of State Fire Marshal.

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

(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. If the average water table elevation is above the pipe springline, proper precautions to avoid flotation must be taken, such as the use of temporary cover, placement of sand or concrete bags on the pipe, or the use of tie downs.
- 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	--
42	42	53	66	93	119	139
48	--	46	58	81	104	--
54	--	--	51	72	93	--
60	--	--	--	65	83	102
66	--	--	--	--	76	93
72	--	--	--	--	70	85
78	--	--	--	--	--	75
84	--	--	--	--	--	--
	3" x 1" Corrugations					
48	--	53	67	93	120	--
54	--	47	59	83	107	--
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	--
90	--	28	35	50	64	78
96	--	--	33	47	60	--
102	--	--	31	44	56	69
108	--	--	--	41	53	65
114	--	--	--	39	50	62
120	--	--	--	37	48	59

Table 856.3C

Corrugated Steel Pipe 2⅔" x ½" Annular Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)				
	Metal Thickness (in)				
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)	0.168 (8 ga.)
18	54	--	--	--	--
21	46	--	--	--	--
24	40	44	--	--	--
30	32	35	--	--	--
36	27	29	38	--	--
42	30	41	65	68	--
48	26	36	57	59	--
54	--	32	50	53	--
60	--	--	45	47	50
66	--	--	--	43	45
72	--	--	--	39	41
78	--	--	--	--	38
84	--	--	--	--	--

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 can be found at the Division of Design, Office of Hydraulics and Stormwater website.

Where cover heights are less than the values shown in the stress reducing slab details, contact the Headquarters Office of Hydraulics and Stormwater Design or the Underground Structures Branch of DES - Structures Design.

Topic 857 – Alternate Materials

857.1 Basic Policy

When two or more materials meet the design service life, and structural and hydraulic requirements, the plans and specifications must provide for alternative pipes, pipe arches, overside drains, and underdrains to allow for optional selection by the contractor. See Index 114.3 (2).

(1) Allowable Alternatives. A table of allowable alternative materials for culverts, drainage systems, overside drains, and subsurface drains is included as Table 857.2. This table also identifies the various joint types described in Index 854.1(1) that should be used for the different types of installations.

(2) Design Service Life. Each pipe type selected as an alternative must have the appropriate protection as outlined in Topic 852 to assure that it will meet the design service life requirements specified in Topic 855. The maximum height of cover must be in accordance with the tables included in Topic 856.

(3) Selection of a Specific Material Type. In the cases listed below, the selection of a specific culvert material must be supported by a complete analysis based on the foregoing factors. All pertinent documentation should be placed on file in the District.

- Where satisfactory performance for a life expectancy of 25 or 50 years, as defined under design service life, cannot be obtained with certain materials by reason of highly corrosive conditions, severe abrasive 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.

way, flow capacity requirements, and the alignment and profile of the highway. Most roadside channels capture sheet flow from the highway pavement and cut slope and convey that runoff to larger channels or to culverts within the drainage system. See Figure 861.2.

Figure 861.2

Roadside Channel Outlet to Storm Drain at Drop Inlet

This initial concentration of runoff may create hydraulic conditions that are erosive to the soil that forms the channel boundary. To perform reliably, the roadside channel is often stabilized against erosion by placing a protective lining over the soil. This chapter presents two classes of channel linings called rigid and flexible linings that are well suited for construction of small roadside channels.

861.2 Hydraulic Considerations

An evaluation of hydraulic considerations for the channel design alternatives should be made early in the project development process. The extent of the hydrologic and hydraulic analysis should be commensurate with the type of highway, complexity of the drainage facility, and associated costs, risks, and impacts. Most of the roadside channels and swales discussed in this chapter convey design flows less than 50 cubic feet per second and generally do not require detailed hydrologic and hydraulic analyses beyond developing the parameters required for the Rational Formula (see Index 819.2(1)), Manning's Equation, and the shear stress equations presented within this Chapter and Hydraulic Engineering Circular (HEC) No. 15, "Design of Roadway Channels with Flexible Linings". The hydraulic design of an open channel consists of developing a channel section to carry the design discharge under the controlling conditions, adding freeboard as needed and determining the type of channel protection required to prevent erosion. In addition to erosion protection, channel linings can be used to increase the hydraulic capacity of the channel by reducing the channel roughness.

The hydraulic capacity of a roadside channel is dependent on the size, shape, slope and roughness of the channel section. For a given channel, the hydraulic capacity becomes greater as the grade or depth of flow increases. The channel capacity decreases as the channel surface becomes rougher. A rough channel can sometimes be an advantage on steep slopes where it is desirable to keep flow velocities from becoming excessively high. See Topics 866 and 867.

(1) *Flood Control Channels.* Flood control channels are typically administered by a local agency and present extreme consequences should failure occur. Therefore, when channels or drainage facilities under the jurisdiction of local flood control agencies or Corps of Engineers are involved, the design must be coordinated via negotiations with the District Hydraulic Engineer and the agencies involved. See Index 861.7, “Coordination with other Agencies” and Index 865.2.

For flood control purposes, a good open channel design within the right of way minimizes the effect on existing water surface profiles. Open channel designs which lower the water surface elevation can result in excessive flow velocities and cause erosion problems. A planned rise in water surface elevation can cause:

- Objectionable flooding of the roadbed and adjacent properties or facilities;
- An environmental and maintenance problem with sedimentation due to reduced flow velocities.

Additional hydraulic considerations may include: movable beds, heavy bedloads and bulking during flood discharges. A detailed discussion of sediment transport and channel morphology is contained in the FHWA’s HDS No. 6 River Engineering for Highway Encroachments. (archived, however, still valid)

Reference is made to Volume VI of the AASHTO Highway Drainage Guidelines for a general discussion on channel hydraulic considerations.

861.3 Selection of "Design Flood"

As with other drainage facilities, the first step in the hydraulic design of roadside channels is to establish the range of peak flows which the channel section must carry. The recommended design flood and water spread criteria for roadway drainage type installations are presented in Table 831.3.

For flood control and cross drainage channels within the right of way, see Index 821.3, “Selection of Design Flood”. Empirical and statistical methods for estimating design discharges are discussed in Chapter 810, "Hydrology".

861.4 Safety Considerations

An important aspect of transportation facility drainage design is that of traffic safety.

The shape of a roadside channel section should minimize vehicular impact and provide a traversable section for errant traffic leaving the traveled way. The ideal channel section, from a traversability standpoint, will have flattened side slopes and a curved transition to the channel bottom. When feasible, it is recommended that channels be constructed outside the clear recovery zone.

Refer to Table 865.2 for typical permitted shear stress and velocity for bare soil and vegetation.

861.10 Lined Channels

The main purposes of channel linings are:

(a) To prevent erosion damage.

(b) To increase velocity for prevention of excessive sedimentation

(c) To increase capacity.

See Topic 865 for design concepts.

861.11 Water Quality Channels

Biofiltration swales are vegetated channels, typically configured as trapezoidal or v-shaped channels (trapezoidal recommended where feasible) that receive and convey stormwater flows while meeting water quality criteria and other flow criteria independent of Chapter 860. Pollutants are removed by filtration through the vegetation, sedimentation, absorption to soil particles, and infiltration through the soil. Strips and swales are effective at trapping litter, total suspended solids (soil particles), and particulate metals. In most cases, flow attenuation is also provided.

Refer to Appendix B, Table B-1 of the Project Planning and Design Guide for a summary of preliminary design factors for biofiltration strips and swales.

See HDM Table 816.6A and Index 865.5 for Manning's roughness coefficients used for travel time calculations for the rational formula based on water quality flow (WQF) to check swale performance against biofiltration criteria at WQF, i.e., a Hydraulic Residence Time of 5 minutes or more; a maximum velocity of 1.0 ft/s; and a maximum depth of flow of 0.5 ft. See Bio-Strips and Bio-Swales under Biofiltration Design Guidance on the Office of Hydraulics and Stormwater Design website.

861.12 References

More complete information on hydraulic principles and engineering techniques of open channel design may be found in FHWA's Hydraulic Design Series No. 4 (HDS 04), "Introduction to Highway Hydraulics", Hydraulic Design Series No. 5 (HDS 05), "Hydraulic Design of Highway Culverts", Hydraulic Engineering Circular No. 15 (HEC 15), "Design of Roadside Channels with Flexible Linings" and Hydraulic Engineering Circular No. 22 (HEC 22), Chapter 5, "Urban Drainage Design Manual – Roadside and Median Channels". For a general textbook discussion of open channel hydraulics, reference is made to "Open-Channel Hydraulics" by Ven Te Chow. In addition, many helpful design aids are included in "Handbook of Hydraulics", by Brater and King. FHWA's HDS and HEC publications are available on FHWA's Hydraulic Publications website.

velocities in unlined channels. Realignment considerations for channels within the right of way are discussed in Index 867, Channel Changes.

862.3 Point of Discharge

The point of discharge into a natural watercourse requires special attention. Water entering a natural watercourse from a highway drainage channel should not cause eddies with attendant scour of the natural watercourse. In erodible embankment soils, if the flow line of the drainage channel is appreciably higher than that of the watercourse at the point of discharge, then the use of a spillway may be advisable to prevent erosion of the channel.

Topic 863 – Channel Section

863.1 Roadside and Median Channels

Roadside and median channels are open-channel systems which collect and convey stormwater from the pavement surface, roadside, and median areas. These channels may outlet to a storm drain piping system via a drop inlet (see Figure 861.2), to a detention or retention basin or other storage component, or to an outfall channel. Roadside and median channels are normally triangular or trapezoidal in cross section and are lined with grass or other protective lining.

Reference is made to the FHWA publication HEC No. 22, Chapter 6.

The shape of a channel section is generally determined by considering the intended purpose, terrain, flow velocity and quantity of flow to be conveyed.

863.2 Triangular

The triangular channel or V-ditch is intended primarily for low flow conditions such as in median and roadside ditches. V-shaped ditches are susceptible to erosion and will require lining when shear stress and velocity exceed the values given for bare soil in Table 865.2. It is good practice to round the bottom of a V-ditch. See Figure 862.1 and Figure 863.1.

863.3 Trapezoidal

The most common channel shapes is the trapezoidal section.

Trapezoidal channels are easily constructed by machinery and are often the most economical.

When a wide trapezoidal section is proposed, both traffic safety and aesthetics can be improved by rounding all angles of the channel cross section with vertical curves. The approximate length of these vertical curves can be determined by the formula:

$$L = \frac{40}{X}$$

where:

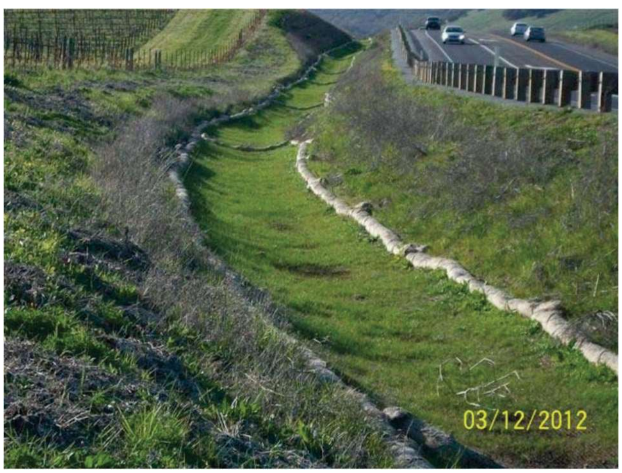
L = Length of vertical curve in feet

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 stormwater 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 (USACE) publication; “Structural Design of Concrete Lined Flood Control Channels” (1995), EM 1110-2-2007, available on the USACE Engineer Manual Publication website.

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.

Trapezoidal Channel Within the Clear Recovery Zone (CRZ): Foreslopes and backslopes of trapezoidal channel constructed within the CRZ should not be steeper than 4:1. Trapezoidal channel sections located within the CRZ should have foreslopes matching the slopes of the CRZ slopes but should not be steeper than 4:1 (refer to Figures 305.6, 307.2, 307.4A, 307.4B, and 307.5). The backslope should not be steeper than 4:1. The bottom width of the channel should not be less than 4 feet (see Figure 834.3). The trapezoidal channel cross-section should satisfy hydraulic conveyance as well as support the load of errant vehicles without the wheels sinking into saturated soil in the channel section. Design criteria for concrete lined channels may be referenced from the US Army Corps Publication "Structural Design of Concrete Lined Flood Control Channels, EM 1110-2-2007".

865.3 Flexible

Flexible linings can be long-term, transitional or temporary. Long-term flexible linings are used where the channel requires protection against erosion for the design service life of the channel. Per Index 861.12, more complete information on hydraulic principles and engineering techniques of flexible channel lining design may be found in HEC No. 15 and Chapter 6 of HEC No. 22.

Flexible linings act to reduce the shear stress on the underlying soil surface. Therefore, the erodibility of the underlying soil is a key factor in the performance of flexible linings. Erodibility of non-cohesive soils (plasticity index less than 10) is mainly due to particle size, while cohesive soil erodibility is a function of cohesive strength and soil density. Vegetative and rolled erosion control product lining performance relates to how well they protect the underlying soil from shear stress, and so these lining types do not have permissible shear stresses independent of soil type. The soil plasticity index should be included in the Materials or Geotechnical Design Report.

In general, when a lining is needed, the lowest cost lining that affords satisfactory protection should be used. This may include vegetation used alone or in combination with other types of linings. Thus, a channel might be grass-lined on the flatter slopes and lined with more resistant material on the steeper slopes. In cross section, the channel might be lined with a highly resistant material (e.g., cellular soil confinement system – see Index 865.3(1) *Long Term*) within the depth required to carry floods occurring frequently and lined with grass above that depth for protection from the rare floods.

(1) Long Term. Long-term lining materials include vegetation, rock slope protection, gabions (wire-enclosed rock), and turf reinforcement mats with enhanced UV stability. Standard Specification Section 72-4 includes specifications for constructing small-rock slope protection for gutters, ditches or channels and includes excavating and backfilling the footing trench, placing RSP fabric and placing small rocks (cobble, gravel, crushed gravel, crushed rock, or any combination of these) on the slope. Where the channel design includes a requirement for runoff infiltration to address stormwater needs, the designer may need to consider installation of a granular filter in lieu of RSP fabric if it is anticipated that the RSP fabric would become clogged with sediment. See following link to HEC No. 23, Volume 2, Design Guideline 16, Index 16.2.1, for information on designing a granular filter:

http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09112/page16.cfm

Standard Specification Section 72-16 includes specifications for constructing gabion structures. Gabions consist of wire mesh baskets that are placed and then filled with rock. Gabion basket wires are susceptible to corrosion and are most appropriate for use as a channel lining where corrosion potential is minimized, such as desert or other arid locations.

Cellular soil confinement systems may be used as an alternative for steep channels with a variety of infills available including soil and gravel. Soil confinement systems consist of sheet polyethylene spot welded to form a system of individual confinement cells. See Figure 865.3.

Figure 865.3

Long-Term Flexible Lining

Placing a polyethylene cellular soil confinement system on a steep-sloped channel.

Per Index 865.1, these systems may be combined with other vegetated flexible linings, e.g., turf reinforcement mats.

(2) *Transitional.* Transitional flexible linings are used to provide erosion protection until a long-term lining, such as grass, can be established. For mild slopes, these may include jute netting (depending on environmental, i.e., wildlife, parameters) or turf reinforcement. Turf reinforcement can serve either a transitional or long-term function by providing additional structure to the soil/vegetation matrix. Typical turf reinforcement materials include gravel/soil mixes and turf reinforcement mats (TRM's). A TRM is a non-degradable rolled erosion control product (RECP) processed into a three-dimensional matrix. For examples see the Headquarters Office of Landscape Architecture's Erosion Control Toolbox website.

The design for transitional products should be based on a flood event with an exceedance probability at least equal to the expected product service life (i.e., 12 to 36 months).

(3) *Temporary.* Temporary channel linings are used without vegetation to line channels that might be part of a construction site or some other short-term channel situation.

Standard Specification Section 21-1 was developed primarily to address slope erosion products, however, it includes specifications for constructing turf reinforcing mats, netting and rolled erosion control products (RECP's – see Index 865.6) which may also be applied to channels as temporary and transitional linings. See Index 865.1for coordinating vegetative recommendation with District Landscape Architecture.

865.4 Composite Lining Design

The procedure for composite lining design is based on the stable channel design procedure presented in Index 864.2 with additional sub-steps to account for the two lining types. Specifically, the modifications are:

Step 1. Determine design discharge and select channel slope and shape. (No change.)

Step 2. Need to select both a low flow and side slope lining. (See Table 866.3A.)

Step 3. Estimate the depth of flow in the channel and compute the hydraulic radius. (No change.)

Step 4. After determining the Manning's n for the low flow and side slope linings, calculate the effective Manning's n:

$$n_e = \left[\frac{P_L}{P} + \left(1 - \frac{P_L}{P}\right)\left(\frac{n_s}{n_L}\right)^{3/2}\right]^{2/3} n_L$$

where:

n_e = Effective Manning's n value for the composite channel

P_L = Low flow lining perimeter, ft

P = Total flow perimeter, ft

n_s = Manning's n value for the side slope lining

n_L = Manning's n value for the low flow lining

Step 5. Compare implied discharge and design discharge. (No change.)

Step 6. Determine the shear stress at maximum depth, τ_d ($\tau_d = \gamma d S$), and the shear stress on the channel side slope, τ_s (see Index 864.2).

Step 7. Compare the shear stresses, τ_d and τ_s, to the permissible shear stress, τ_p, for each of the channel linings. If τ_d or τ_s is greater than the τ_p for the respective lining, a different combination of linings should be evaluated. See Table 865.2.

865.5 Bare Soil Design and Grass Lining

Per Index 865.1, the District Landscape Architect should be contacted to recommend vegetation alternatives (including vegetation for transitional products, if needed) and the same procedure for the stable channel design procedure presented in Index 864.2 should be followed by the Project Engineer. See Figure 865.4 for grass lining example in a median channel. For slope stability when constructing embankment (4:1 and steeper), 85-90% relative compaction is desired. Although not optimal for best plant growth, compaction of up to 90% is not a major constraint for grass establishment. Prior to seeding, scarification to a depth of 1 inch of the compacted soil surface is recommended for improving initial runoff absorption and ensuring the seed is incorporated into the soil. A temporary degradable erosion control blanket (ECB) (e.g., single net straw) can then be installed on top.

The permissible shear stress for the vegetation is based on the design flood (Table 831.3). If the calculated shear for any given vegetation method is inadequate, then an alternative vegetation type with greater shear stress must be selected and/or a different channel shape may be selected that results in a lower depth of flow.

Figure 865.4

Grass-Lined Median Channel

The permissible shear stress for rolled erosion control products should be based on a flood event with an exceedance probability no less than the expected product service life (i.e., 12 to 36 months). The maximum shear stresses for channel applications shown in Erosion Control Technology Council Rolled Erosion Control Products Specification Chart must be lower than the permissible shear stresses indicated in Table 865.2. See the Erosion Control Technology Council's Specification website at: https://www.ectc.org/specifications.

The Manning's roughness coefficient for grass linings varies depending on grass properties and shear stress given that the roughness changes as the grass stems bend under flow. The equation describing the n value for grass linings is:

$$n = \alpha C_n \tau_0^{-0.4}$$

where:

τ_0 = Average boundary shear stress, lb/ft^2

α = Unit conversion constant, 0.213

C_n = Grass roughness coefficient (use 0.20 or Tables 4.3 and 4.4 from HEC-15)

The remaining shear at the soil surface is termed the effective shear stress. When the effective shear stress is less than the allowable shear for the soil surface, then erosion of the soil surface will be controlled. The effective shear at the soil surface is given by the following equation.

$$\tau_e = \tau_d \left(1 - C_f\right) \left(\frac{n_s}{n}\right)^2$$

where:

τ_e = Effective shear stress on the soil surface, lb/ft^2

Table 865.2

Permissible Shear and Velocity for Selected Lining Materials[2] (cont.)

Boundary Category	Boundary Type	Permissible Shear Stress (lb/ft^2)	Permissible Velocity (ft/s)
Rolled Erosion Control Products (RECPs)			
Temporary Degradable Erosion Control Blankets (ECBs)	Single net straw	1.65	3
	Double net coconut/straw blend	1.75	6
	Double net shredded wood	1.75	6
Open Weave Textile (OWT)	Jute	0.45	2.5
	Coconut fiber	2.25	4
	Vegetated coconut fiber	8	9.5
	Straw with net	1.65	3
Non Degradable Turf Reinforcement Mats (TRMs)	Unvegetated	3	7
	Partially established	6.0	12
	Fully vegetated	8.00	12
Rock Slope Protection, Cellular Confinement and Concrete			
Rock Slope Protection	Small-Rock Slope Protection (4-inch Thick Layer)	0.4	5
	Small-Rock Slope Protection (7-inch Thick Layer)	0.8	6
	Class I	2.4	10
	Class III	4.8	12
Gabions	Gabions	6.3	12
Cellular Confinement: Vegetated infill	71 in^2 cell and TRM	11.6	12
Cellular Confinement: Aggregate Infill	1.14 - in. D_{50} (45 in^2 cell)	6.9	12
	3.5” D_{50} (45 in^2 cell)	15.1	11.5
	1.14” D_{50} (71 in^2 cell)	13.2	12
	3.5” D_{50} (71 in^2 cell)	18	11.7
	1.14” D_{50} (187 in^2 cell)	10.92	12
	3.5” D_{50} (187 in^2 cell)	10.55	12
Cellular Confinement: Concrete Infill	(71 in^2 cell)	2	12
Hard Surfacing	Concrete	12.5	12

NOTES:
[1]PI = Plasticity Index (From Materials or Geotechnical Design Report)

[2]Some materials listed in Table 856.2 have been laboratory tested at shear stresses/velocities above those shown. For situations that exceed the values listed for roadside channels, contact the District Hydraulic Engineer.

- *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 eight 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 16, Highways in the River Environment: Roads, Rivers, and Floodplains (2023)
- HEC 18, Evaluating Scour at Bridges (2012)
- HEC 20, Stream Stability at Highway Structures (2012)
- HEC 23, Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance (2009)
- HEC 25, Highways in the Coastal Environment (2020)
- HEC 26, Culvert Design for Aquatic Organism Passage (2010)

(b) AASHTO Highway Drainage Guidelines – General guidelines for good erosion control practices are covered in Volume III - Erosion and Sediment Control in Highway Construction

(c) 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.

(d) U.S. Army Corps of Engineers EM 1110-2-1601 Hydraulic Design of Flood Control Channels Manual.

(e) California Department of Fish and Wildlife – California Salmonid Stream Habitat Restoration Manual.

(f) 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 and Investigation.

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 HEC’s 16,18, 20, and 23 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).

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 Stormwater 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 FHWA's HEC16 Highways in the River Environment.

(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

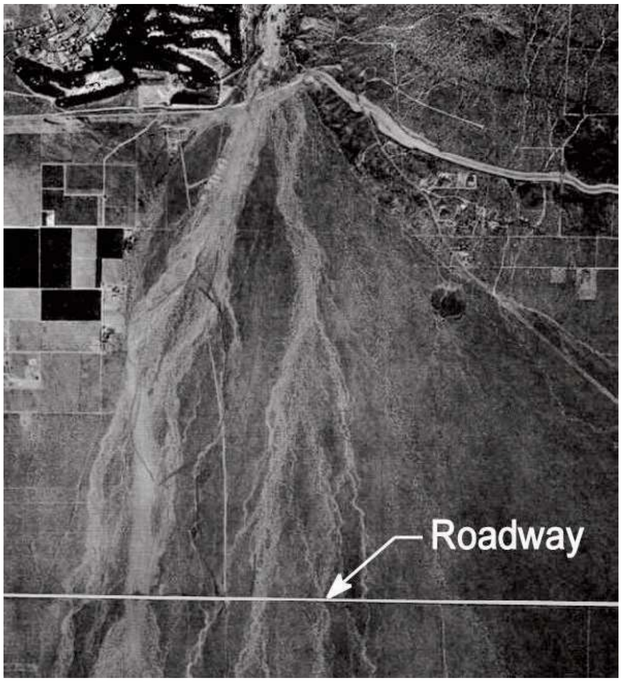
Only with such knowledge can the project engineer develop the necessary arguments to make the case that erosion control measures must be designed to avoid significant deterioration of the stream environment not only in the immediate vicinity of the highway encroachment or crossing, but in many instances 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

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" available at the FEMA.gov website.

(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, found on the California Department of Fish and Wildlife website.

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. 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 Stormwater 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 Hydraulics and Stormwater 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), see Index 873.6 for bridge design criteria.

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 7 in, HEC 20 and Caltrans Sediment Transport and Fluvial Geomorphology at Crossing Structures Design Guidance, 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, also known as Matrix Riprap
- 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 outcropping of erosion-resistant materials, trees, shrubs, or other indications of stability.

- 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 website for the Defense Technical Information Center, 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

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 available on the Division of Design website.

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.

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 23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 8, for further discussion on the use of articulated concrete blocks.

(4) Rigid Revetments.

(a) Concreted-Rock Slope Protection.

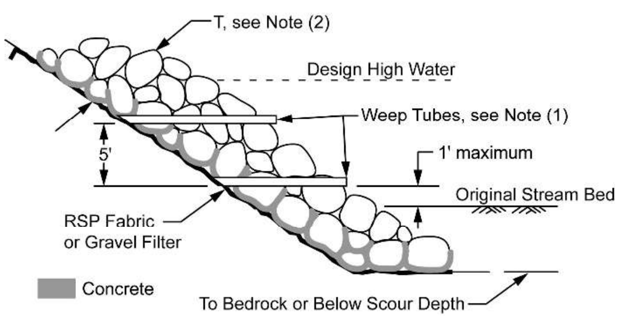
(1) General Features. This type of revetment consists of rock slope protection with interior voids filled with PCC to form a monolithic armor. A typical section of this type of installation is shown in Figure 873.3E.

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), also known as Matrix Riprap, 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 Hydraulics and Stormwater 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.

(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 Chapter 8 of HEC 23 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:

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 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(4), Construction, Easements, Access and Staging, and Index 872.3(5), 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.

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 200-year flood. This 200-year scour countermeasure design flood standard is independent of the 50-year design flood used by the District for protecting the channel bank or the bridge approach embankment (see Index 873.2). See Index 827 for culvert energy dissipators.

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

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:

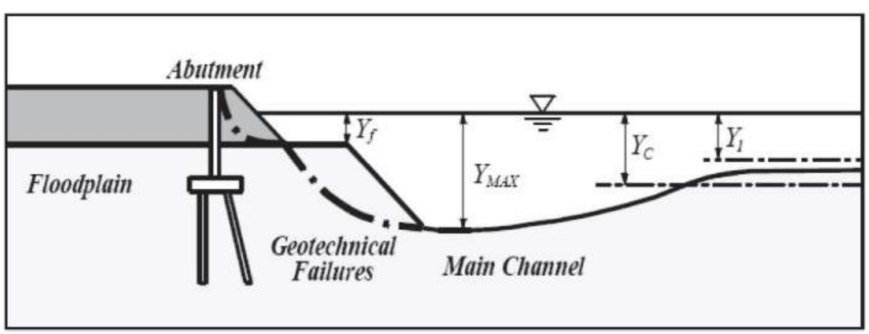
- Scour potential of channel material.
- Natural erosion potential of stream banks that may affect project features. See Figure 873.6C.
- The performance of existing cut, fill and natural slopes including the slope soil/rock composition.
- Slope stability analysis and need for earth retaining systems including gabion walls.
- Embankment constructability and impact to nearby structures or bridge abutments. Refer to the Geotechnical Manual and Figure 873.6D, found on the Caltrans Engineering Services (DES) website.

Figure 873.6C

Lateral Stream Migration Within a Canyon Setting Example

Figure 873.6D

Conceptual Geotechnical Failures Resulting from Abutment Scour



CHAPTER 880 – SHORE PROTECTION

Topic 881 – General

Index 881.1 – Introduction

Highways, bikeways, pedestrian facilities and appurtenant installations are often constructed along lakes and coastal zones. These locations are under attack from the action of waves and may require protective measures.

Shore protection along coastal zones and lake shores that are subjected to wave attack can be a major element in the design, construction, and maintenance of highways. Chapter 880 deals with procedures, methods, devices, and materials commonly used to mitigate the damaging effects of wave action on transportation facilities and adjacent properties. The primary focus is on quantifying exposure of these locations to sea level rise, storm surge, and wave action. Coastal engineering involves complex physical processes, sometimes too complex for adequate theoretical description, and the design level of risk is often high.

Refer to Index 806.2 for definitions of drainage terms.

881.2 Design Philosophy

In each district there should be a designer or advisor, usually the District Hydraulic Engineer, knowledgeable in the application of shore protection principles and the performance of existing works at coastal and lake shore locations vulnerable to wave attack.

Information is also available from headquarters specialists in the Division of Design and Structures Design in the Division of Engineering Services (DES). The most effective designs result from involvement with Design, Environmental, Landscape Architecture, Structures, Construction, and Maintenance (for further discussion on functional responsibilities see Topic 802). For habitat characterization and assessment relative to design and obtaining project specific permits, the designer may also require input from biologists. The District Hydraulic Engineer will typically be able to

assist with selecting storm scenarios for design wave heights, the design of high water level (including sea or lake level change estimates) using coastal surge and wave models, flood analysis, water surface elevations/profiles, shear stress computations, scour analysis and hydraulic analysis for placement of coastal structures.

There are a number of ways to deal with the wave action and shore erosion.

- Where avoidance is not feasible, the simplest way and generally the surest of success and permanence, is to locate the facility away from the erosive forces. This is not always feasible or economical, but should be the first consideration. Locating the facility to higher ground or solid support should never be overlooked, even when it requires excavation of solid rock, since excavated rock may serve as a valuable material for protection at other points of attack.
- The most commonly used method is to armor the shore with a more resistant material like rock slope protection. FHWA Hydraulic Engineering Circular 25 (HEC 25), Third

Edition, presents general issues and approaches in coastal highway design. Types of revetments for wave attack and coastal structures are covered in Chapters 7 and 8.

- *Rock Materials*. Optimum use should be made of local materials, considering the cost of special handling. Specific gravity and gradation of stone are major factors in shore protection and the specified minimum should not be lowered without increasing the mass of stones. See Index 873.3(3)(a)(2)(b) for equations to estimate rock size.
- *Nature*-based hybrid coastal protection solutions can be used as a protection strategy for highways.

881.3 Selected References

Hydraulic and drainage related publications are listed by source under Topic 807. References specifically related to shore protection measures are listed here for convenience.

(a) FHWA Hydraulic Engineering Circulars (HEC) – The following 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 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 (2020)

(b) “FHWA Nature-Based Solutions for Coastal Highway Resilience: An Implementation Guide” (2019) provides information on where and how nature based solutions can be used to protect roadway infrastructure. These techniques can also be used for shoreline protection in certain scenarios. These techniques are more appropriate in bays, estuaries, or other areas without high energy waves, therefore, nature based solutions are only appropriate in very limited areas. Hybrid techniques can combine nature based and structural techniques, to meet both goals.

(c) AASHTO Highway Drainage Guidelines – General guidelines for good erosion control practices are covered in Chapter 3 – Erosion and Sediment Control in Highway Construction; and Chapter 11 - Guidelines for Highways along Coastal Zones and Lakeshores.

(d) AASHTO Drainage Manual (2014) – Refer to Chapters; 12 – Energy Dissipators; 18 – Channel and Stream Bank Stabilization ; and 19 – Coastal Zone; and 20 – Erosion and Sediment Control. This document supersedes the 2005 AASHTO Model Drainage Manual and provides guidance on engineering practice in conformance with FHWA’s HEC and HDS publications and other nationally recognized engineering policy and procedural documents.

(e) USACE Shore Protection Manual (SPM) (1984) – Comprehensive two volume guidance on wave and shore processes and methods for shore protection. No longer in publication but still referenced to provide wave equations and potential screening criteria.

(f) USACE Design of Coastal Revetments, Seawalls, and Bulkheads. Engineering Manual 1110-2-1614 (1995) – Supersedes portions of Volume 2 of the Shore Protection Manual (SPM).

(g) USACE Coastal Engineering Manual. Engineer Manual (EM) 1110-2-1100 (2002) – Published in six parts plus an appendix, this set of documents supersedes the SPM and EM 1110-2-1614.

(h) Caltrans Design Manual for Hybrid Coastal Protection Strategies (2022) – Provides design guidance on analysis for coastal protection structures, hybrid strategies, nature-based strategies, design recommendations and examples.

Topic 882 – Planning and Location Studies

882.1 Planning

The development of sustainable, cost effective and environmentally friendly protective works requires careful planning and a good understanding of both the site location, local-land use and development, and habitat within the shore or coastal zone subject to wave attack. Planning begins with an office review followed by a site investigation.

Google Earth can be a useful tool for determining site location and recent changes to the coastal zone.

Nearby bridges should be reviewed for site history and changes in stream cross-section. All bridge files belong to Structure Maintenance within the Division of Maintenance.

Coastal highways traverse bays, estuaries, beaches, dunes and bluffs which are some of the most unique and treasured habitats for humans as well as the habitats of a variety of plants and animals. The list of endangered species requiring these coastal habitats for survival includes numerous sea turtles, birds, mammals, rodents, amphibians and fish. District biologist staff should be consulted early on during the project planning phase for subject matter expertise to perform an initial habitat assessment.

For habitat characterization and preliminary assessment relative to design and obtaining project specific permits, the initial site investigation team should include the project engineer, the district hydraulic engineer, and a biologist.

The selection of the type of protection can be determined during or following site investigation. For some sites the choice is obvious; at other sites several alternatives or combinations may be applicable.

Considerations at this stage are:

- Design life and whether the protection need be permanent or temporary.
- The severity of wave attack.
- The coastal water level and future sea level.
- Littoral drift of the beach sands.
- Seasonal shifts of the shore.
- The ratio of cost of highway replacement versus cost of protection.
- Analysis of foundation and materials explorations.
- Access for construction
- Slope (H:V)
- Vegetation type and location
- Physical habitat

- Failure mode (see Table 872.2)
- Total length of protection needed
- Distance to an alternative route

The second step is the selection and layout of protective elements in relation to the highway facility.

882.2 Site Consideration

The determination of the lengths, heights, alignment, and positioning of the protection is 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) *Lakes and Tidal Basins.* Highways adjacent to lakes or basins may be at risk from wave generated erosion. All bodies of water generate waves. Height of waves is a function of fetch and depth. Erosion along embankments behind shallow coves is reduced because the higher waves break upon reaching a shoal in shallow water. The threat of erosion in deep water at headlands or along causeways is increased. Constant exposure to even the rippling of tiny waves may cause severe erosion of some soils.

Older lakes normally have thick beds of precipitated silt and organic matter. Bank protection along or across such lakes must be designed to suit the available foundation. It is usually more practical to use lightweight or self-adjusting armor types supported by the soft bed materials than to excavate the mud to stiffer underlying soils. See Index 883.3 for further information on armor protection.

In fresh waters, effective protection can often be provided by the establishment of vegetation, but planners should not overlook the possibility of moderate erosion before the vegetative cover becomes established. A light armor treatment should be adequate for this transitional period.

(2) *Ocean Front Locations.* Wave action is the erosive force affecting the reliability of highway locations along the coast. The corrosive effect of salt water is also a major concern for hydraulic structures located along the coastline. Headlands and rocks that have historically withstood the relentless pounding of tide and waves can usually be relied on to continue to protect adjacent highway locations founded upon them. The need for shore protection structures is, therefore, generally limited to highway locations along the top or bottom of bluffs having a history of sloughing and along beach fronts.

Beach protection considerations include:

- Attack by waves.
- Littoral drift of the beach sands.
- Seasonal shifts of the shore.
- Foundation for protective structures.

Wave attack on a beach is less severe than on a headland, due to the gradual shoaling of the bed which trips incoming waves into a series of breakers called a surf

Littoral drift of beach sands may either be an asset or a liability. If sand is plentiful, a new beach will be built in front of the highway embankment, reducing the depth of water at its toe and the corresponding height of the waves attacking it. If sand supply is less plentiful or subject to seasonal variations, the new beach can be induced or retained by revetments to develop pocket beaches.

If sand is in scant supply, backwash from a revetment tends to degrade the beach or bed even more than the seasonal variation, and an allowance should be made for this scour when designing the revetment, both as to weight of stones and depth of foundation. Groins may be ineffective for such locations; if they succeeded in trapping some littoral drift, downcoast beaches would recede from undernourishment.

Seasonal shifts of the shoreline result from combinations of:

- Ranges of tide.
- Reversal of littoral currents.
- Changed direction of prevailing onshore winds.
- Attack by swell.

Generally the shift is a recession, increasing the exposure of beach locations to the hazard of damage by wave action. On strands or along extensive embayments, recession at one end may result in deposition at the other. Observations made during location assessment should include investigation of this phenomenon. For strands, the hazard may be avoided by locating the highway on the backshore facing the lagoon.

Foundation conditions vary widely for beach locations. On a receding shore, good bearing may be found on soft but substantial rock underlying a thin mantle of sand. Bed stones and even gravity walls have been founded successfully on such foundations. Spits and strands, however, are radically different, often with softer clays or organic materials underlying the sand. Sand is usually plentiful at such locations, subsidence is a greater hazard than scour, and location should anticipate a "floating" foundation for flexible, self-adjusting types of protection.

In planning ocean-front locations, the primary decision is a choice of (1) alignment far enough inshore to avoid wave attack, (2) armor on the embankment face, or (3) off shore devices like revetments to aggrade the beach at embankment toe.

Topic 883 – Design

883.1 Introduction

A set of plans and specifications must be prepared to define and describe the protection that the design engineer has in mind. See Index 873.1.

Recommendations on slope protection, and erosion control materials can be requested from the District Hydraulic Engineer, the District Materials Branch, the Office of Geotechnical Services, and the Office of Hydraulic and Stormwater 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.

883.2 Design High Water, Design Wave Height and Sea Level Rise

Information needed to design shore protection is:

- Design High Water Level
- Design Wave Height

(1) Design High Water Level

Designs should not be based on an arbitrary storm, high tide or flood frequency.

Per Index 873.2, a suggested starting point to provide an initial screening step is to determine the design high water level to ensure the protection withstands high water levels caused by meteorological conditions having a recurrence interval of one-half the service life of the protected facility. Depending on the type of facility, it may 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 to conform with a subsequent analysis which considers the level of related risks, local historic high water marks, sea level rise and climate change. Scour countermeasures protecting structures designed by the Division of Engineering Services (DES) should include consideration of floods greater than a 1 percent probability of exceedance (100-year frequency of recurrence). See Index 873.6.

There is always some risk associated with the design of protection features. Significant 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.

(a) Lake Shore Locations. The flood stage elevation on a lake or reservoir is usually the result of inflow from upland runoff. If the water stored in a reservoir is used for power generation, flood control, or irrigation, the design high water elevation should be based on the owner’s schedule of operation.

(b) Coastal Locations.

Except for inland tidal basins effected by wind tides, floods and seiches, the static or still-water level used for design of shore protection is the highest tide. In tide tables, this is the stage of the highest tide above "tide-table datum" at Mean Lower Low Water (MLLW). NOAA provides information on datum conversions at select tide gage locations. For locations without a gage, you can use VDATUM to make datum conversions. The online software is available at: https://vdatum.noaa.gov/vdatumweb/

To clarify the determination of design high-water, Fig. 883.2A shows the *Highest Tide* in its relation to an extreme-tide cycle and to a hypothetical average-tide cycle, together with nomenclature pertinent to three definitions of tidal range. Note that the cycles have two highs and two lows. The average of all the higher highs for a long period (preferably in multiples of the 19-yr. metonic cycle) is Mean Higher High Water (MHHW), and of all the *lower* lows, MLLW. The vertical difference between them is the *diurnal range*.

Particularly on the Pacific coast where MLLW is datum for tide tables, the stage of MHHW is numerically equal to diurnal range.

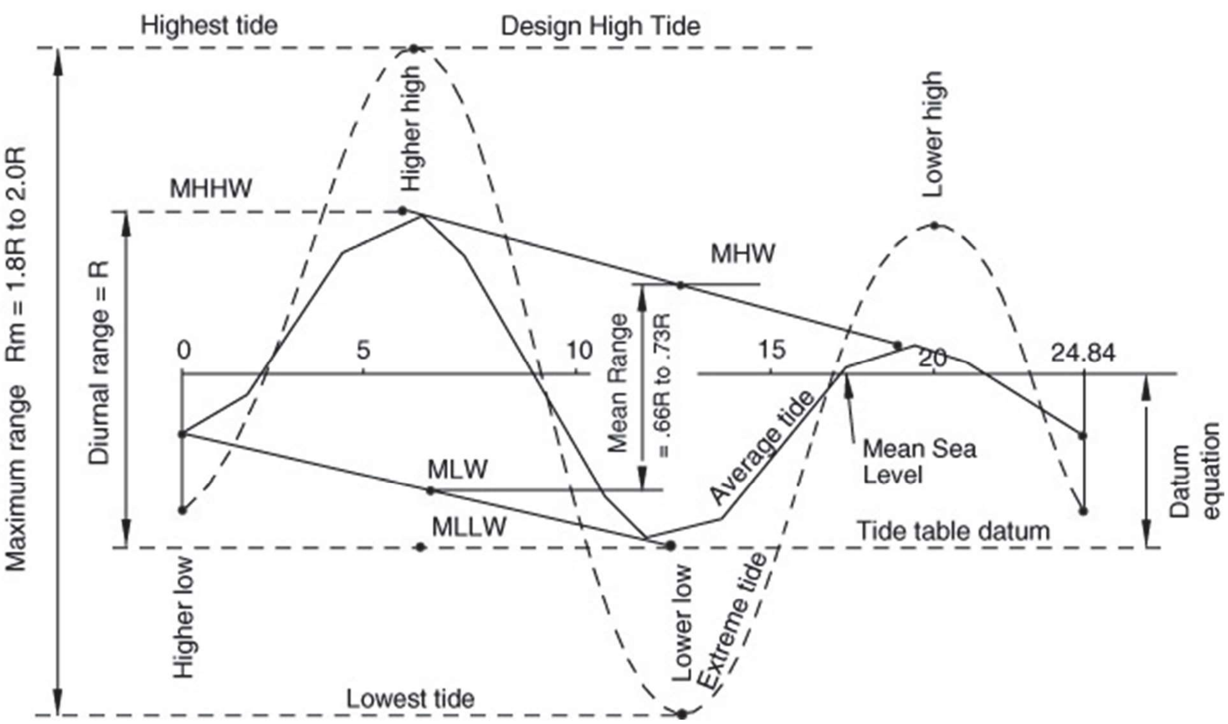
The average of all highs (indicated graphically as the mean of higher high and lower high) is the MHW, and of all the lows, MLW. Vertical difference between these two stages is the *mean range*.

See the Surveys Manual and the interim survey guidelines for Estimating Sea Level. For more information, see the Office of Land Surveys website.

See Index 814.5, Tides and Waves, for information on where tide and wave data may be obtained. See HEC 25, for a discussion on tidal and survey datums.

Figure 883.2A

Nomenclature of Tidal Ranges



NOTES:

(1) Because of the great variation of tidal elements, Figure 883.2A was not drawn to scale.

(2) The elevation of the design high tide may be taken as mean sea level (MSL) plus one-half the maximum tidal range (Rm).

(2) Design Wave Heights.

(a) General. Even for the simplest of cases, the estimation of water levels caused by meteorological conditions is complex. For preliminary design or low risk coastal design, simplified techniques as described below may be used. Project designs with medium to high risk should use computer models as described in HEC 25, Index 5.4 and the Design Manual for Hybrid Coastal Protection Strategies. Simplified techniques may be used to predict acceptable wind wave heights for the design of highway protection facilities along the shores of embayments, inland lakes, and reservoirs. The Coastal Engineering Manual provides a simplified wave prediction method which is suitable for most riprap sizing applications.

The method is described in HEC 23, Volume 2, Index 17.2.2 of Design Guideline 17. It is recommended that for ocean shore protection designs the assistance of FHWA or a certified coastal engineer be requested.

Shore protection structures are generally designed to withstand the wave that induces the highest forces on the structure over its economic service life. The design wave is analogous to the design storm considerations for determining return frequency. A starting point of reference for shore protection design is the maximum significant wave height that can occur once in about 20-years. Economic and risk considerations involved in selecting the design wave for a specific project are basically the same as those used in the analysis of other highway drainage structures.

(b) Wave Distribution Predictions. Wave prediction is called hindcasting when based on past meteorological conditions and forecasting when based on predicted conditions. The same procedures are used for hindcasting and forecasting. The only difference is the source of the meteorological data. Reference is made to the Army Corps of Engineers, Coastal Engineering Manual – Part II, and the Design Manual for Hybrid Coastal Protection Strategies for more complete information on the theory of wave generation and predicting techniques.

The prediction of wave heights from boat generated waves must be estimated from observations. Some Boussinesq wave models also contain capabilities to estimate wave heights generate from boats.

The surface of any large body of water will contain many waves differing in height, period, and direction of propagation. A representative wave height used in the design of bank and shore protection is the significant wave height, H_S. The significant wave height is the average height of the highest one-third of all the waves in a wave train for the time interval (return frequency) under consideration. Thus, the design wave height generally used is the significant wave height, H_S, for a 20-year return period.

Other design wave heights can also be designated, such as H_{10} and H_1. The H_{10} design wave is the average of the highest 10 percent of all waves, and the H_1 design wave is the average of the highest 1 percent of all waves. The relationship of H_{10} and H_1 to H_S can be approximated as follows:

$H_{10} = 1.27\ H_S$ and $H_1 = 1.67\ H_S$

Economics and risk of catastrophic failure are the primary considerations in designating the design wave average height.

(c) Wave Characteristics. Wave height estimates are based on wave characteristics that may be derived from an analysis of the following data:

- Wave gage records
- Visual observations
- Published wave hindcasts
- Wave forecasts
- Maximum breaking wave at the site

(d) Predicting Wind Generated Waves. The height of wind generated waves is a function of fetch length, windspeed, wind duration, and the depth of the water.

(1) Hindcasting – The U.S. Army Corp of Engineers has historical records of onshore and offshore weather and wave observations for most of the California coastline. Design wave height predictions for coastal shore protection facilities should be made using this information and hindcasting methods.

Deep-water ocean wave characteristics derived from offshore data analysis may need to be transformed to the project site by refraction and diffraction techniques. As mentioned previously, it is strongly advised that the Corps technical expertise be obtained so that the data are properly interpreted and used.

(2) Forecasting – Simplified wind wave prediction techniques may be used to establish probable wave conditions for the design of highway protection on bays, lakes and other inland bodies of water. Wind data for use in determining design wind velocities and durations is usually available from weather stations, airports, and major dams and reservoirs. The following assumptions pertain to these simplified methods:

- The fetch is short, 75 miles or less
- The wind is uniform and constant over the fetch.

It should be recognized that these conditions are rarely met and wind fields are not usually estimated accurately. The designer should therefore not assume that the results are more accurate than warranted by the accuracy of the input and simplicity of the method. Good, unbiased estimates of all wind generated wave parameters should be sought and the cumulative results conservatively interpreted. The individual input parameters should not each be estimated conservatively, since this may bias the result.

The applicability of a wave forecasting method depends on the available wind data, water depth, and overland topography. Water depth affects wave generation and for a given set of wind and fetch conditions, wave heights will be smaller and wave periods shorter if the wave generation takes place in transitional or shallow water rather than in deep water.

The height of wind generated waves may also be fetch-limited or duration-limited. Selection of an appropriate design wave may require a maximization procedure considering depth of water, wind direction, wind duration, wind speed, and fetch length.

Procedures for predicting wind generated waves are complex and our understanding and ability to describe wave phenomena, especially in the region of the coastal zone, is limited. Many aspects of physics and fluid mechanics of wave energy have only minor influence on the design of shore protection for highway purposes. Designers interested in a more complete discussion on the rudiments of wave mechanics should consult the U.S. Army Corps of Engineers' Coastal Engineering Manual – Part II.

An initial estimate of wind generated significant wave heights can be made by using Figure 883.2B. If the estimated wave height from the nomogram is greater than 2 feet, the procedure may need to be refined. Refer to the Design Manual for Hybrid Coastal Protection Strategies or HEC-25 for recommended calculation methods for larger wave heights. It is recommended that advice from the FHWA or a coastal engineer be obtained to refine significant wave heights, Hs, greater than 2 feet.

(e) Breaking Waves. Wave heights derived from hindcasts or any forecasting method should be checked against the maximum breaking wave that the design stillwater level depth and nearshore bottom slope can support. The design wave height will be the smaller of either the maximum breaker height or the forecasted or hindcasted wave height.

The relationship of the maximum height of breaker which will expend its energy upon the protection, Hb, and the depth of water at the slope protection, ds, which the wave must pass over are illustrated in Figure 883.2C.

The following diagram, with some specific references to the SPM, summarizes an overly simplified procedure that may be used for highway purposes to estimate wind generated waves and establish a design wave height for shore protection.

(f) Wave Run-up. Run-up is the extent, measured vertically, that an incoming wave will rise on a structure. An estimate of wave run-up, in addition to design wave height, will typically be needed and is required by policy for projects subject to California Coastal Commission (CCC) jurisdiction Techniques for calculating wave runup on permeable and impermeable structures can be found in Hydraulic Response and Armor Layer Stability on Coastal Structures (2015) See the University of Delaware's Center for Applied Coastal Research CACR Reports website at:

https://coastal.udel.edu/research-2/cacr-reports/.

Additional guidance on procedures for estimating wave run-up for rough surfaces (e.g., RSP) are contained in the U.S. Army Corps of Engineers manual, Design of Coastal Revetments, Seawalls, and Bulkheads, (EM 1110-2-1614) published in 1995.

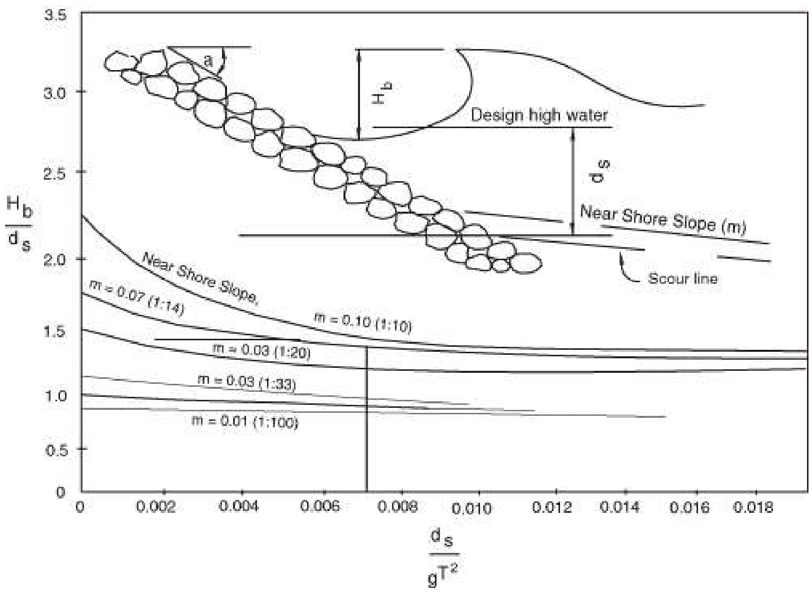
Procedures for estimating wave run-up for smooth surfaces (e.g., concrete paved slopes) and for vertical and curved face walls are contained in the U.S. Army Corps of Engineers, Shore Protection Manual, 1984. See Figure 873.2D for estimating wave run-up on smooth slopes for wave heights of 2 feet or less.

In protected bays and estuaries, waves generated by recreational or commercial boat traffic and other watercraft may dominate the design over wind generated waves. Direct observation and measurements during high tidal cycles may provide the designer the most useful tool for establishing wave run-up for these situations.

(g) Littoral Processes. See Index 882.3(2). Littoral processes result from the interaction of winds, waves, currents, tides, and the availability of sediment. The rates at which sediment is supplied to and removed from the shore may cause excessive accretion

Figure 883.2C

Design Breaker Wave



Example:

By using hindcast methods, the significant wave height (H_s) has been estimated at 4 feet with a 3 second period. Find the design wave height (H_d) for the slope protection if the depth of water (d) is only 2 feet and the nearshore slope (m) is 1:10.

Solution:

$$\frac{d_s}{gT^2} = \frac{2\ ft}{(32.2\ ft/sec^2) \times (3\ sec)^2} = 0.007$$

From Graph - $H_b/d_s = 1.5$

$H_b = 2 \times 1.5 = 3.0$ ft

Answer:

Since the maximum breaker wave height, H_b, is smaller than the significant deepwater wave height, H_s, the design wave height H_d is 3.0 feet.

T = Wave Period (SPM)

or erosion that can affect the structural integrity of shore protection structures or functional usefulness of a beach. The aim of good shore protection design is to maintain a stable shoreline where the volume of sediment supplied to the shore balances that which is removed. Designers interested in a more complete discussion on littoral processes should consult the U.S. Army Corps of Engineers' Coastal Engineering Manual (CEM) – Part III.

(3) *Sea Level Rise.* The California Ocean Protection Council (OPC) has developed sea-level rise guidance for use by state and local governments to assess the associated risks with sea-level rise and incorporate sea-level rise into planning, permitting and investment decisions. The “State of California Sea-Level Rise Guidance 2024 Science and Policy Update” provides estimates of sea-level rise based upon the best available science. The sea level rise scenarios are derived from probabilistic projections developed in the Intergovernmental Panel on Climate Change Sixth Assessment report (IPCC AR6). A step by step approach to selecting a value for sea-

level rise based on OPC 2024 guidance is provided in the steps below. More detail on each step is available in the 2024 OPC guidance. This method of evaluating sea-level rise could be revised and updated in the future based on the most current guidance provided by OPC or other responsible agencies. Routine maintenance projects are exempt from Sea Level Rise analysis requirements (i.e., pavement replacement, culvert replacement, etc.).

Step 1. Identify the nearest tide gauge. The rates of sea-level rise along the California coast is dependent on land elevations resulting from tectonic activity as well as land subsidence. There are 14 active tide gauges along the California coast and sea-level rise projections vary across the tide gauges based on trends in tectonic activity and land subsidence. The tide gauges incorporate the localized effects of vertical land motion.

Identify the tide gauge nearest to the project site. If the project is located equidistant between two tide gauges it would be appropriate to interpolate between the two gauges or average the two gauges. The 14 tide gauges along the California coast are identified in Figure 883.2 D.

Step 2. Evaluate planning and/or project time horizon(s). Determine the project lifespan (design life) for selection of appropriate year for associated sea-level rise. The California Transportation Commission has adopted asset classes associated with the State Highway System and the Primary Asset Classes are defined as: (a) Pavement, (b) Bridges, (c) Drainage, (d) Transportation Management Systems, and (e) Supplementary Assets. In the absence of a designated project lifespan, the design life associated with an asset class should be used to determine the year associated with the projected sea-level rise. Design lives of pavement projects are referenced in Section 612, and maintenance free service life of culverts (typically 50-years) referenced in chapter 850 of this manual. Bridge Design Life (per AASHTO LRFD Bridge Design Specifications 9th Edition Sec. 1.2) is 75 years.

Lifespan (design life) will help focus the decision of the appropriate sea level rise scenario. The OPC 2024 update shows a greater certainty in the sea level rise expected in the next 30 years compared with the sea level rise projections in the 2018 OPC guidance. In the mid-term (2050-2100) there is a large range of sea level rise as there is greater uncertainty in the projected future warming from different emission pathways and certain physical processes such as rapid ice sheet melt. The range of sea level rise over the long term (2100 and beyond) becomes increasingly large due to uncertainties associated with physical processes. Once the design life of the project location is complete, reevaluate at that time using the best available science.

Step 3. Choose multiple Sea Level Rise Scenarios for vulnerability assessment. Vulnerability of people, communities, natural resources, infrastructure and properties should be considered for developing a range of sea-level rise projections. There are five Sea Level Rise scenarios: Low, Intermediate-Low, Intermediate, Intermediate-High, and High. The H++ Scenario (from previous guidance) has been determined as not physically plausible and is not included in the 2024 OPC report. The Low Scenario is the lower bound for the plausible sea level rise in 2100 with aggressive emission reduction scenarios. The Intermediate-Low Scenario is the reasonable lower bound for the most likely range of sea level rise by 2100. With this scenario, low confidence processes such as ice sheet melt are not included. The Intermediate Scenario is driven dominantly by high emission scenarios and is a reasonable estimate of the upper bound of most likely sea level rise in 2100. Low confidence processes contribute about 25% of the pathways for reaching this scenario. The Intermediate-High Scenario reflects intermediate to high future emissions and high warming where rapid ice sheet loss processes (low confidence processes) are contributing to sea level rise. The High Scenario reflects both high emissions and low confidence processes. This scenario relies on large contributions from rapid ice-sheet loss and processes where there is low confidence in their understanding.

For exceedance probabilities of each scenario see Table 2.2 in the 2024 OPC guidance document.

FHWA and others recommend looking at multiple climate change scenarios to determine the appropriate scenario for the project or asset. Consideration should be given to risk, the criticality of the asset, stranded assets, adaptive capacity, cost and other economic, engineering, social and environmental concerns. As there is uncertainty in sea level rise projections and climate change scenarios, it can be practical to choose a lower sea level rise scenario for design and develop an adaptation plan for when and if the higher scenario occurs. Adaptation plans allow for flexibility in design and address uncertainties in sea level rise projections. Jurisdictional agencies (such as the California Coastal Commission) may require consideration of sea-level rise projections under a High Scenario; however project design may not necessarily include incorporation of the highest value of sea-level rise selected. Factors such as project costs and feasibility, may require a negotiated agreement with the agencies to develop a modular approach to design using a value associate with a shorter time frame than the selected design life of the project with the understanding that successive projects over time would build upon the proposed design to ultimately provide a resilient infrastructure.

Figure 883.2D

California 14 Tide Gauges

For low risk facilities, using lower sea level rise scenarios such as Low or Intermediate-Low is warranted. Examples may include a parking lot within the coastal area, or a constructed trail leading down towards a beach. Should such assets be damaged or destroyed, they may be relatively easy to repair or replace.

Intermediate-Low or Intermediate sea level rise scenarios may be considered for medium risk projects. Such risk may be exercised for a segment of roadway that if inaccessible would not jeopardize public safety or public health. Additionally, such a risk may be adopted if an asset would be cost effective to repair/replace as opposed to major resiliency redesign, and whose inaccessibility would not negatively impact natural resources or properties. Another example may be culvert outfalls that may tend to be inundated by sea-level rise on a coastal highway. The Intermediate-Low or Intermediate Scenario may be assumed if a contingency plan exists to retrofit the culvert outfalls with tidal flap gates to prevent backflow.

Intermediate or Intermediate-High Scenarios may be considered when an asset is expected to be needed for public health and safety. The likelihood that sea-level rise may meet or exceed this value is low. A highway expected to be used as an emergency evacuation route for people/communities, as access to and from hospitals, as a major route for support of local/regional economies, may be evaluated for sea-level rise under the Intermediate-High Scenario.

High risk projects can consider the Intermediate-High or High Scenarios and may be used for projects that have little to no adaptive capacity, that are essential for public safety and health, that is cost prohibitive to replace or repair, and with a design life well beyond 2050. An example would be a major bridge connecting communities with access to hospitals and economic interests and spanning a water body directly impacted by sea-level rise, and where freeboard requirements are necessary for passage of ships, boats or other crafts. Such situations with project design lives extending into the 22nd century where a minimum freeboard is required for passage of watercraft may require consideration of the High Scenario.

Sea-level rise projections for each tide gauge are provided in the "State of California Sea Level Rise Guidance 2024 Update" Appendix 1 and may be accessed at:

https://opc.ca.gov/wp-content/uploads/2024/05/Item-4-Exhibit-A-Final-Draft-Sea-Level-Rise-Guidance-Update-2024-508.pdf

Step 4. Conduct vulnerability assessment. Evaluate impacts from projected sea level rise scenarios, including exposure, sensitivity, and adaptive capacity. This step includes looking at exposure maps of sea level rise inundation and flooding, erosion, and groundwater considerations. It is important to consider impacts to the asset. Adaptive capacity is the ability of an asset to evolve in response to or cope with the impacts of sea level rise. Evaluate potential impacts and adaptive capacity across a range of sea-level rise projections. Evaluate the potential impacts of sea-level rise on the project in terms of vulnerable communities, critical infrastructure, and economic burden.

Step 5. Explore adaption options and feasibility: Explore project-specific adaptation options and possibly a cost-benefit analysis considering physical, economic, environmental, social and legal constraints. Evaluate impacts of sea-level rise by using sea-level rise mapping tools (sea-level rise) viewer available at:

https://coast.noaa.gov/slr/#/layer/slr/0/13566681.667176013/4585243.78640795/9/satellite/none/0.8/2050/interHigh/midAccretion

NOAA's sea-level rise viewer evaluates the impacts of sea-level rise at water surface elevations derived from adding the selected value of sea-level rise to MHHW elevation of the sea in the vicinity of the project. MHHW values for various stations may be obtained from:

https://tidesandcurrents.noaa.gov/stations.html?type=Datums

Select appropriate station and datum from website. Add selected value of sea-level rise to MHHW to obtain water surface elevation for design.

Step 6: Select phased adaptation approach and/or implement project. Select sea-level rise projections based on risk tolerance and incorporate appropriate resiliency into design. Adaptation plans may be included in case sea-level rise exceeds design projections. Consider risk, budget, regulatory constraints, environmental and community impacts and stakeholder input. An example for selection of sea-level rise for two hypothetical projects near Crescent City, Del Norte County is provided below.

Example Project # 1 **:** An HM pavement project is to be constructed south of Crescent City. The project will include cold planing and repaving the existing highway. Sea level rise considerations do not need to be evaluated on this project as it is a routine maintenance project.

Example Project # 2 : A segment of SR 101 is to be reconstructed south of Crescent City. A parking lot for access to the beach is also included in the scope of the project as shown in Figure 883.2E.

Assumed project scope includes reconstruction of segment of SR-101 south of Crescent City. A parking lot is to be constructed for beach access for recreational purposes. Consideration of sea-level rise for proposed project is as follows:

The nearest tide gauge is Crescent City. The data for sea-level rise at Crescent City is provided in Table 883.1B. Per HDM Index 612.2, for roadside facilities such as a parking lot, 20 year design life may be used, while the pavement design life for new construction or reconstruction projects should be 40 years. Applicable sea-level rise for the parking lot will be for year 2040 (20 years from the project design date). Applicable sea-level rise for roadway reconstruction will be for year 2060 (40 years from the project design date). Districts have flooding records which can be used to determine if sea level rise or flooding is an issue that needs to be addressed.

Consider range of sea-level rise for varying risk and scenarios. For the parking lot, sea-level rise for projects prior to 2050 are described as near term and show much greater certainty in the amount of sea level rise expected. Refer to Table 883.1B for information:

- Sea-level rise associated with a Low Scenario for year 2040 is 0.1 feet.
- Sea-level rise associated with an Intermediate-Low Scenario for the year 2040 is 0.2 feet.
- Sea-level rise associated with the Intermediate Scenario for the year 2040 is also 0.2 feet as there is not much difference in the near term for different sea level rise scenarios.

Figure 883.2E

Crescent City Example

Now evaluate the impact of the potential loss of the parking lot.

- The loss of the parking lot is not expected to have a significant impact on public health and safety. The loss would be expected to have an insignificant economical impact on any community.
- When there is no significant economical loss, no threat to public safety, public health or transportation resulting from the loss of the parking lot, an evaluation of the costs of construction, repair and replacement should determine the risk factor to be adopted for selection of an appropriate value for sea-level rise.
- Although sea-level rise associated with a Low scenario may be justified, costs of construction, future right of way issues, future repair or replacement should be examined. For the parking lot the differences between sea-level rise values associated with the Low scenario, Intermediate-Low Scenario and the Intermediate Scenario is very small (ranges from just over 1 inch to almost 2.5 inches) and based on costs an appropriate scenario may be selected.

For highway reconstruction, sea-level rise projections for projects with design life extending beyond 2050 fall under the mid-term range where the differences become increasingly large and are more closely associated with potential future greenhouse gas emissions. With a 40-year design life for pavement reconstruction projects sea-level rise for year 2060 may be considered. Refer to Table 883.1B for information. Review sea-level rise projections for multiple scenarios. The comparisons for year 2060 are provided Table 883.1A.

Table 883.1A

Crescent City Example Comparison for 2060

Low Scenario	Intermediate-Low Scenario	Intermediate Scenario
0.1 foot	0.4 foot	0.6 foot

Table 883.1B

Sea-Level Scenarios Crescent City

YEAR	LOW	INT-LOW	INTERMEDIATE	INT-HIGH	HIGH
2020	0.1	0.1	0.1	0.1	0.1
2030	0.1	0.1	0.2	0.2	0.2
2040	0.1	0.2	0.2	0.3	0.4
2050	0.1	0.3	0.4	0.6	0.8
2060	0.1	0.4	0.6	1.0	1.5
2070	0.2	0.4	0.8	1.6	2.3
2080	0.2	0.6	1.2	2.3	3.4
2090	0.2	0.7	1.7	3.0	4.5
2100	0.2	0.8	2.3	3.9	5.6
2110	0.2	0.9	2.9	4.7	6.9
2120	0.2	1.0	3.4	5.3	7.9
2130	0.2	1.2	3.8	5.8	8.7
2140	0.2	1.3	4.2	6.3	9.6
2150	0.2	1.4	4.7	6.8	10.3

Determine impact of potential loss of this segment of highway on communities:

- Will the loss of this segment impact transport of patients to and from a hospital?
- Will the loss of this segment impact response times for emergency vehicle?
- Will the loss of this segment impact freight and deliveries resulting in economic losses?
- Can traffic be detoured easily around this segment?

The value of sea-level rise for designing the roadway may be selected after evaluating relevant issues such as mentioned above.

If the highway segment is important for public safety, health and local economy, the Intermediate or Intermediate-High Scenario's value of 0.6 or 1.0 feet respectively, may be selected. The design would not only incorporate a higher elevation of the roadway but would also include measures for protecting the roadway (Armoring sea approach to roadway embankment).

Although the project does not have to be designed for the High Scenario, it may be considered. Sea-level rise associated with the High Scenario is 1.5 feet. A plan for future modular adaptation may be included should it become apparent at some time in future that sea levels are heading towards the High Scenario projections. The plan may include raising the profile of the highway and associated protection measures against the 1.5 feet projected sea-level rise.

Determine MHHW elevation from NOAA's Tides and Currents website for the Crescent City, CA Datum at:

https://tidesandcurrents.noaa.gov/datums.html?id=9419750.

Figure 883.2F shows the results.

Add MHHW to projected sea-level rise:

For the roadway project add 1.0 feet to 6.87 feet. Elevation of water surface including sea level rise associated with the Intermediate-High Scenario is 7.87 feet. Evaluate impact of sea-level rise on project by using NOAA sea-level rise viewer at: https://coast.noaa.gov/slr/.

(4) *Assessing Extreme Events and Climate Change.* Chapter 14 of HEC 25, presents guidance on specific methodologies for assessing exposure of coastal transportation infrastructure to extreme events and climate change. For all projects, as a minimum, the use of existing data and resources should be utilized through the use of existing inundation (FEMA) or tsunami hazard maps to determine the exposure of infrastructure under selected sea (lake) level change scenarios, and sensitivity to depth-limited wave or wave runup processes. See HEC 25, Indexes 14.2 and 14.6.1 Level of Effort 1: Pacific Coast – Storms.

883.3 Coastal Protection

(1) *General.* For Caltrans, coastal protection is divided into two categories; hard shoreline protection and nature-based hybrid coastal protection strategies. Nature-based hybrid strategies merge structural shore protection devices with nature-based coastal protection solutions. Executive Orders B-30-15 and N-82-20 obligate State of California agencies to prioritize and accelerate the use of natural infrastructure solutions. Armor is the artificial surfacing of shore or embankment to resist erosion or scour. Armor devices can be flexible (self-adjusting) or rigid. The distinction between revetments (layers of rock or concrete), seawalls, and bulkheads is one of functional purpose. Revetments usually consist of rock slope protection on the top of a sloped surface to protect the underlying soil. Seawalls are walls designed to protect against large wave forces. Bulkheads are designed primarily to retain the soil behind a vertical wall in locations with less wave action. Design issues such as tie-backs, depth of sheets are primarily controlled by geotechnical issues. The use of each one of the three types of coastal protection depends on the relationship between wave height and fetch (distance across the water body). Bulkheads are most common where fetches and wave heights are small. Seawalls are most common where fetches and wave heights are large. Revetments are often common in intermediate situations such as on bay or lake shorelines.

(2) *Revetments.*

(a) Rock Slope Protection (RSP). Hard armoring of shorelines, primarily with 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.

For RSP designs along coastal and lake shores, for wave heights five feet or less, the methodology presented in HEC 23, Volume 2, Design Guideline 17- Riprap Design for Wave Attack has been formally adopted by the Caltrans Bank and Shore Protection Committee. See also HDM Chapter 870, HEC-23 and HEC-25 for additional guidance on RSP. Section 72 of the Standard Specifications provides all construction and material specifications.

Rock is usually the most economical type of revetment where stones of sufficient size and quality are available. It also has the following advantages:

- Wave run-up is less than with smooth types (See Figure 883.2G).
- It is salvageable, may be stockpiled and reused if necessary.

In designing the rock slope protection for a shore location, the following determinations are to be made for the typical section.

- Depth at which the stones are founded (bottom of toe trench. See Figure 883.2I and Figure 17.2 in HEC 23, Volume 2, Design Guideline 17).
- Elevation at the top of protection.
- Rock size, specific gravity and section thickness.
- Need for geotextile or rock filter material.
- Face slope.

Well designed coastal rock slope protection should:

- Assure stability and compatibility of the protected shore as an integral part of the shoreline as a whole.
- Not be placed on a slope steeper than 1.5H:1V.
- Use stone of adequate weight to resist erosion, derived from Index 883.3(2)(a)(2)(1).
- Prevent loss of bank materials through interstitial spaces of the revetment. Rock slope protection fabric or a filter layer 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).
- Be constructed of rock of such shape as to form a stable protection structure of the required section. See Index 873.3(3)(a)(2)(a).

(1) General Features – See Index 873.3(3)(a)(1)(a) through (e) for discussions on methods of placement, foundation treatment, rock slope protection fabrics and gravel filters.

(2) Stone Size – Two methods for determining riprap size for stability under wave action are presented in HEC 23, Volume 2, Design Guideline 17: (1) the Hudson method, and (2) the Pilarczyk method.

(a) The Hudson Method. Applications of Hudson's equation in situations with a design significant wave height of H=5 feet or less have performed well. This range of design wave heights encompasses many coastal revetments along highway embankments. When design wave heights get large and the design water depths get large, problems with the performance of rubble-mound structures can occur. A more conservative design approach should use a more conservative H statistic. The proper input wave height statistic is required and discussed in Section 7.3 of HEC 25. RSP with design wave heights much greater than H=5 feet require more judgment and more experience and input from a trained, experienced coastal engineer. Therefore, when design wave heights are much greater than H=5 feet, contact the District Hydraulic Engineer. The Hudson method considers wave height, riprap density, and slope of the bank or shoreline to compute a required weight of a median-size riprap particle.

$$W_{50} = \frac{\gamma_r H^3 (\tan\theta)}{K_d (S_r - S_w)^3}$$

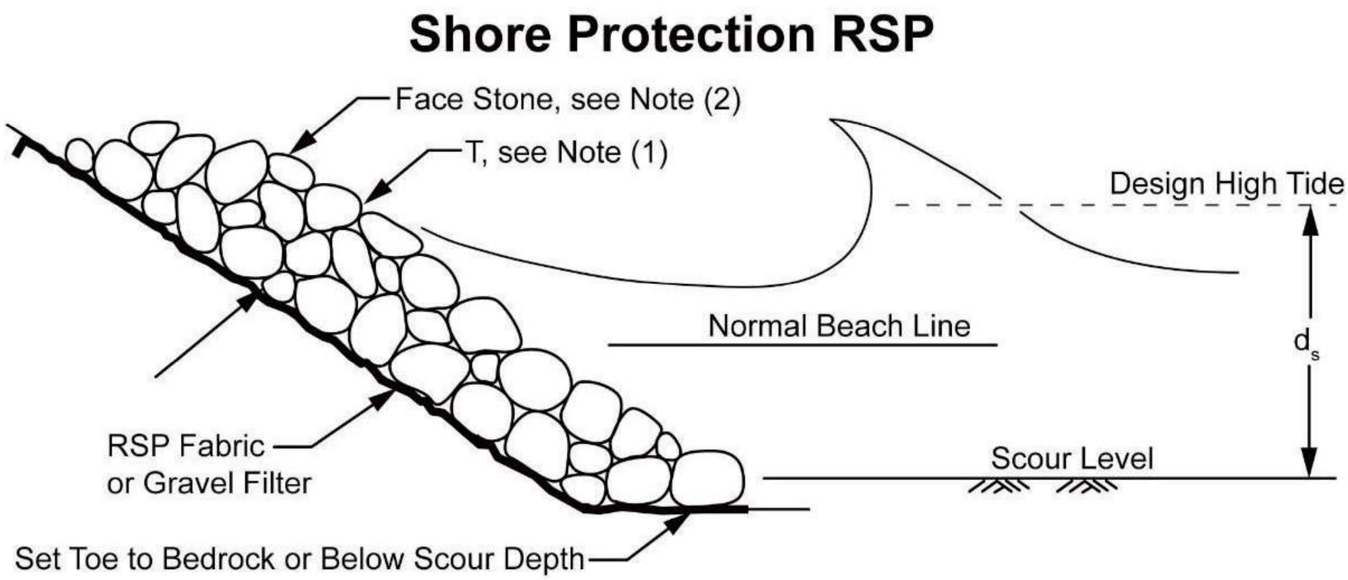
Where:

W_{50} = weight of median riprap particle size, (lb)

γ_r = unit weight of riprap, (lb/ft^3)

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)(a)(2).

(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 counter forted 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 7.1 of HEC 25, 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.

As stated in HEC-25, groins were probably the most common shoreline stabilization technique in the first half of the 20th century. However, they are much less acceptable today due to their potential downdrift negative impacts. Such negative impacts can be mitigated through use of modern coastal engineering principles to appropriately address downdrift impacts when used in combination as a hybrid solution. Refer to Design Manual for Hybrid Coastal Protection Strategies for detailed groin design guidance.

(6) *Nature-Based Hybrid Coastal Protection.* This type of coastal protection incorporates soft nature-based techniques with traditional hard protection methods. The soft engineering measures provide an environmentally friendly component that can enhance ecological connectivity, while the hard measures provide resistance and protection against the effects of extreme tidal and storm events as well as sea level rise. The use of nature-based hybrid protection strategies will satisfy directions presented in California Executive Orders B-30-15 and N-82-20 to consider climate change adaptation strategies and promote coastal resiliency through the use of natural infrastructure solutions.

Given the high energy, varying tidal events, and dynamic coastline of California, nature-base hybrid protection strategies are preferred over stand-alone nature-based strategies. The hard protection components is needed to protect the coastline and infrastructure from vulnerabilities during extreme events caused by sea level rise, wave runup and overtopping, king tide events, and tsunamis. While the soft components can provide attenuation of wave energy during smaller magnitude tidal and storm events, their bigger benefit is to improve ecology and habitat along the coastline.

Suitable soft components of nature-based hybrid protection for the California coastline are beach nourishment and dune construction. By placing site-specific beach sediment to replenish the sediment supply, the retreating shorelines in California can be reestablished and widened that will create a buffer zone and protect adjacent infrastructure from wave attack. Dune construction on the beach will also protect infrastructure from wave attack by physically attenuating the wave energy. If it is site-appropriate, dunes and bluffs can be planted to further improve the ecology and habitat.

Regarding hard component examples of a nature-based hybrid protection strategy, rock can be placed at the toe of a slope or a bluff along a beach at proper depth to resist scour of the toe and possible failure of a slope or bluff. The rock along the toe of slope can be hidden beneath beach sediment. Similarly, rock can be hidden in the inner core of a dune. The exterior shell of the dune would be comprised of beach sediment. If wave forces exceed the resistance capacity of the sediment outer shell, the interior rock can provide resistance and continue the wave energy attenuation to protect the shoreline and adjacent infrastructure. This would prevent complete failure of the dune and allow for its continued function.

The nature-based hybrid protection adaptation strategy combines the positive benefit of ecological preservation with the positive benefit of extreme event protection to provide needed resiliency for the California coastline. Refer to the Caltrans *Design Manual for Hybrid Coastal Protection Strategies* for design and analysis guidance as well as maintenance considerations for nature-based hybrid coastal protection.

CHAPTER 890 – STORMWATER MANAGEMENT

Topic 891 – General

Index 891.1 – Introduction

The term “stormwater management” refers to the cooperative efforts of public agencies and the private sector to mitigate, abate, or reverse the adverse results, both in water quantity and water quality, associated with the altered runoff phenomena that typically accompanies urbanization. Stormwater management encompasses a number of control measures, which may be either structural or non-structural (including policy and procedural measures) in nature.

The State Water Resources Control Board (SWRCB) has the authority under the Clean Water Act and Porter-Cologne Act to issue National Pollutant Discharge Elimination System (NPDES) permits. Caltrans has a robust stormwater management program which focuses on NPDES permit compliance. This chapter will introduce the permits and provide resources to incorporate the necessary control measures in project development.

This chapter will focus primarily on the management of stormwater runoff quantity, and NPDES permit compliance. Information related to the designer’s responsibility for the management of stormwater runoff quality is contained in the Department’s Project Planning and Design Guide (PPDG).

891.2 Philosophy

When runoff impacts result from a Department project, then the cost of mitigating these impacts is a legitimate part of the project cost. Since transportation funds are increasingly limited, and because mitigation of runoff problems can be expensive, it is important to identify the causative factors and responsible parties. When runoff impacts are caused by others, avenues for assigning these costs to the responsible party should be evaluated. The local agencies responsible for land use in the area are a good place to begin this evaluation, as many of these local agencies have enacted land use regulations in an effort to control flooding. These regulations often require that developers limit changes in the volume and rate of discharge between the pre- and post-development site conditions. In addition, many local agencies must be responsive to their own stormwater permits which require that they implement programs to control the quality of stormwater discharges within their jurisdiction. When runoff impacts are caused jointly by the Department and others, it may be possible to develop cooperative agreements allowing joint impact mitigation. See Indexes 803.2 and 803.3 for further discussion on cooperative agreements and up-grading of existing highway drainage facilities.

891.3 Permits

Federal regulations for controlling discharges of pollutants from Multiple Separate Storm Sewer Systems (MS4s), construction sites, and industrial activities were incorporated into the NPDES permit process by the 1987 amendments to the Clean Water Act and by the subsequent 1990 promulgation of federal stormwater regulations issued by the U.S. EPA. The EPA regulations require municipal, construction, and industrial stormwater discharges to comply with an NPDES permit. In California, the EPA delegated its authority to issue NPDES permits to the SWRCB.

In 1970, the Porter-Cologne Water Quality Control Act took effect and created nine Regional Water Quality Control Boards (RWQCBs). While the SWRCB writes the statewide NPDES permits and has jurisdiction throughout California, the RWQCBs enforce them. The RWQCBs may also adopt region-specific permits. See Section 3.2.1 in the PPDG or the SWRCB website for a map of RWQCB jurisdiction and links to each RWQCB for a list of region-specific requirements.

The Caltrans stormwater management program has been developed to comply with the following NPDES permits:

(1) *Caltrans NPDES Statewide Stormwater Permit (Caltrans Permit)*. This is a MS4 permit to regulate stormwater and non-stormwater discharges specifically from Caltrans properties and facilities, and discharges associated with operation and maintenance of the State highway system. The Caltrans Permit applies to all work on Caltrans right-of-way by Caltrans, Local Agencies, and Encroachment Permit recipients.

The Caltrans Stormwater Management Plan describes how Caltrans plans to implement the Caltrans Permit requirements. It describes Caltrans' stormwater management program and addresses stormwater pollution control related to various activities, such as planning, design, construction, maintenance and operations of roadways and facilities, and presents key implementation responsibilities and schedules.

Under the 2022 Caltrans Permit, highway projects in the state right-of-way creating 10,000 square feet or more of New Impervious Surface (NIS), or non-highway facility projects creating 5,000 square feet or more of NIS trigger a requirement to implement post-construction treatment BMPs. The PPDG provides directions to calculate NIS, a list of approved treatment BMPs, and a checklist to select the most appropriate treatment BMPs for the project.

The Statewide Trash Provision were incorporated into the 2022 Caltrans Permit and added to the Caltrans Stormwater Management Plan. The discharge of trash to surface waters of the State is prohibited by the Statewide Trash Provisions which was included as an attachment to the 2022 Caltrans Permit. Caltrans has developed a Statewide Trash Implementation Plan to ensure compliance with the trash provisions which delineates Significant Trash Generation Areas (STGAs) within Caltrans jurisdiction. Projects with a treatment requirement, in accordance with the Caltrans

Permit, located within a STGA must install certified full-capture trash post-construction treatment BMPs where feasible. Refer to the PPDG section 1.4.2.2 for more information on trash provisions.

(2) *Construction General Permit (CGP)*. The SWRCB elected to adopt a single statewide general permit for construction activities that typically applies to stormwater discharges from sites with soil disturbance of one (1) or more acres. Projects triggering CGP coverage are required to prepare and implement a site-specific Stormwater Pollution Prevention Plan (SWPPP) to identify and manage potential sources of stormwater pollution from construction sites. The SWPPP would include water pollution control drawings showing locations of BMPs selected to best protect pollutants from discharging from the site. Refer to the PPDG Chapter 3 for BMP types and applicability. This statewide CGP applies to all of California except for projects located on Tribal Lands or within the Lake Tahoe Hydrologic Unit.

(3) *Lake Tahoe CGP*. Projects located in the Lake Tahoe Hydrologic Unit must work under the Lake Tahoe CGP issued by the Lahontan RWQCB. It typically applies to stormwater discharges from construction activities with one (1) or more acres of soil disturbance, similar to the statewide CGP, however it holds different requirements targeted for Lake Tahoe as the receiving water. Projects working under the Lake Tahoe CGP are required to prepare a site-specific SWPPP to comply with permit requirements.

(4) *U.S. EPA CGP*. Projects located on Federal Tribal Lands must work under the Federal CGP issued by the U.S. EPA. It typically applies to stormwater discharges from construction activities with one (1) or more acres of soil disturbance. The tribal entity or program may have additional stormwater specific requirements to comply with and should be coordinated with during the design phase, however most projects on tribal reservations utilize the U.S. EPA CGP. Projects working under the U.S. EPA CGP will have to prepare a site-specific SWPPP to comply with permit requirements.

(5) *Industrial General Permit (IGP)*. The IGP regulates industrial stormwater discharges and authorized non-stormwater discharges from industrial facilities in California. The IGP regulates discharges associated with 9 federally defined categories of industrial activities, such as a batch plant or crushing plants which deliver to more than one construction site. The IGP has a different set of personnel requirements and an industrial site-specific SWPPP. If needed, permit is typically obtained by the Contractor for Caltrans projects, but Caltrans has oversite and approval rights.

891.4 Design Standards

During preparation of the project plans, it is not always possible to know where a contractor will perform certain activities. To provide the contractor with flexibility, but to assure that proper controls are implemented, the Construction Contract Standards cover most jobsite stormwater measures. This ensures that stormwater measures will be implemented for certain activities regardless of where on the site those activities are performed. Standard Plan Sheets T51 through T67 provide Temporary Water Pollution Control details. Although Division I of the Standard Specifications requires contractors to comply with all permits, Section 13 of the Standard Specifications has been established to clarify permit requirements specific to work done in Caltrans right-of-way. There are a series of special provisions to assist with common project-specific needs, such as working under an erosivity waiver, working on tribal lands or in the Lake Tahoe Hydrologic Unit, street sweeping limitations, etc. Coordinate with the District Stormwater Design and Construction Stormwater Coordinators for a list of bid items associated with the Section 13 Standard Specifications and guidance to prepare the engineers estimate.

Section 62 of the Standard Specifications provides construction requirements for many of the approved treatment Best Management Practices as discussed in Topic 892.2. Contact the Headquarters Office of Hydraulics and Stormwater Design for nonstandard specification needs.

Topic 892 – Stormwater Management Strategies

892.1 General

Quantity / Quality Relationship. Management of stormwater quality often requires the assessment of relatively small runoff producing events. As much as 80 percent of average annual rainfall is produced by storms with return periods of less than 2 years. As a result, water quality facilities are typically sized to address relatively small runoff volumes. Conversely, stormwater quantity management is typically directed at reducing the peak flow rate on storms with a 10-year or greater return period, and water quantity control facilities must be sized accordingly.

The Caltrans Permit requires that the stormwater runoff water volumes used for sizing treatment BMPs be based on the 85^{th} percentile, 24-hour storm while full-capture trash devices are sized to treat runoff generated by the 1-year,1-hour event. See PPDG Section 5 for treatment BMP sizing calculation requirements.

In order to achieve both water quantity and quality benefits, it may be necessary to use a combination of strategies or control measures. For example, placement of a relatively small detention basin or filtration immediately upstream of a quantity attenuating detention basin can provide sediment capture, while allowing larger flows to be mitigated by the major basin. Some types of water quality control measures will need to incorporate bypass features so that the smaller, more frequent, runoff events can be treated while still allowing larger flows to be routed away from the traveled way. Refer to the Headquarters Office of Hydraulics and Stormwater Design website for treatment BMP design guidance documents to be utilized to site and size treatment BMPs.

892.2 Types of Strategies

There are various stormwater management strategies which may be used to mitigate the effects of stormwater runoff problems. They vary from very simple to very complex techniques depending upon specific site conditions and regulatory requirements which must be satisfied.

The PPDG provides both design guidance on specific water quality control measures as well as a more general discussion of how and when to incorporate water quality control measures into projects. Additional Stormwater Quality Handbooks that go into more detail on Construction Site BMPs include the *Construction Site Best Management Practice (BMP) Manual*, *Stormwater Pollution Prevention Plan (SWPPP)*, *Water Pollution Control Program (WPCP) Preparation Manual*, as well as Maintenance BMP guidance in the *Maintenance Staff Guide*. See standard plans and Section 13 of the standard specifications regarding CGP and temporary BMPs to be implemented for compliance with SWPPP and WPCP.

In addition to the measures described in the PPDG, the following measures may provide relief in dealing with the water quantity side of stormwater management.

(1) *Best Management Practices (BMPs).* The PPDG provides an overview of different types of BMPs and when they should be considered. BMPs can be broken into four categories: Design Pollution Prevention BMPs, Treatment BMPs, Construction Site BMPs and Maintenance BMPs, as shown in the table below. Refer to Section 3.3 in the PPDG for an overview of BMPs.

BMP	**Description**	**Responsible Division for BMP implementation**
Design Pollution Prevention (DPP) BMPs	Permanent soil stabilization and concentrated flow controls and slope protection systems, etc.	Design, Construction, and Maintenance
Treatment BMPs	Permanent treatment devices and facilities	Design, Construction, and Maintenance
Construction Site BMPs	Temporary soil stabilization and sediment control, non-stormwater management, waste management, etc.	Design and Construction
Maintenance BMPs	Litter pickup, drainage cleaning, street sweeping, etc.	Maintenance

Design guidance for specific Treatment BMPs, such as biofiltration strips/swales, detention devices, infiltration areas, media filters, etc., provide information related to design elements, calculations, cost estimates and layout details. Refer to the Office of Hydraulics and Stormwater Design website for references and resources.

(2) *Groundwater Impacts.* In some locations highly permeable underground strata may allow percolation of excess runoff into the ground. Benefits include recharge of underground aquifers and the possible reduction or elimination of conveyance systems along with pollutant removal. Special care must be exercised in areas of high groundwater to avoid potential contamination of the aquifer.

- *SGMA. The Sustainability Groundwater Management Act (SGMA)* was passed in 2014 which defines sustainable groundwater management as the "management and use of groundwater in a manner that can be maintained during the planning and implementation horizon without causing undesirable results." Undesirable results include:

- Chronic lowering of groundwater levels indicating a significant and unreasonable depletion of supply
- Significant and unreasonable reduction of groundwater storage
- Significant and unreasonable sea water intrusion
- Significant and unreasonable degraded water quality
- Significant and unreasonable land subsidence
- Depletion of interconnected surface water that have significant and unreasonable adverse impacts on beneficial uses of the surface water

SGMA required local agencies to form groundwater sustainability agencies then develop and implement basin specific groundwater sustainability plans to avoid the undesirable results listed above and mitigate overdraft within 20 years. The Department of Water Resources provides regulatory oversight of this program, more information can be found on their website: water.ca.gov/sgma

Caltrans projects with the potential to impact groundwater basins will need to be aware of local groundwater sustainability agencies' regulations and coordinate with them to avoid contributing to undesirable results.

- *Drinking water.* If a project has the potential to impact groundwater, special care must be taken to avoid impacts to drinking water wells. Typically, local city or county health departments issue well drilling permits and maintain well logs which are publicly available. The SWRCB's Groundwater Ambient Monitoring and Assessment Program has additional resources and maintains an online database of statewide well logs from the *Well Completion Report Map Application*.
- *Injection Wells.* The Environmental Protection Agency (EPA) has additional regulations on Class V Wells, which are used to inject non-hazardous fluids underground. Injection wells pose a threat to ground water quality if not managed properly. Most Class V wells are unsophisticated shallow systems that depend on gravity to drain fluids into the ground (e.g., stormwater drainage wells, dry wells, etc.), but there are over 20 well subtypes that fall into the Class V category.

  The EPA established minimum requirements to prevent injection wells from contaminating underground sources of drinking water. EPA Region 9 has enforcement responsibility for injection wells in California. More information and permitting details can be found on the EPA Class V Wells website.
- *Waste Discharge Requirements.* The Porter Cologne Act requires a report of waste of waste discharge requirements (ROWD) with the applicable RWQCB to construct injection wells to protect groundwater from discharges. It is recommended to discuss a proposed project that could potentially impact groundwater quality with the RWQCB before submitting the ROWD.

(3) *Drainage Easements*. In areas where right of way is inexpensive it may be possible to purchase flood easements. These areas are typically used for agriculture and are subject to flooding at any time during specified times of the year. Cooperative agreements with local agencies or flood control districts will typically be necessary.

892.3 Design Considerations

The items presented below describe some of the issues to be considered prior to, and during, the design of any stormwater management facility. General issues common to most stormwater management strategies that need to be evaluated are:

- Access for maintenance must be provided, and the facility must be maintainable. Stormwater control facilities must not become regarded as wetlands themselves, which would require special permits for routine maintenance.
- Facilities should be designed to "blend in" with their surroundings to the greatest extent possible. The district landscape architecture unit should be contacted for assistance.
- The effects of the proposed facility on channel capacities and existing floodways require evaluation. Care must be taken to evaluate the effects related to the delayed release from detention facilities since an increase in downstream peak discharges may result (see Figure 892.3).
- The effects of releasing sediment free "hungry" water into channels and the potential for increased erosion rates downstream must be determined.
- Evaluate the effects of depriving downstream water users (human, aquatic or vegetative) of runoff due to retention, percolation or other diversion.
- Avoid creating vector habitat by introducing permanent pools of water unless concurrence is obtained from the local vector control agency. Typically, pools of water left longer than 96-hoiurs can provide vector habitat.

Stormwater management techniques involving on-site and off-site storage may offer the highway design engineer the more reasonable and responsive solution to problems relative to the handling of excess runoff. The cooperation of other jurisdictions is generally a prerequisite to applying these strategies and a cooperative agreement is almost always necessary. See Chapter 12 of the AASHTO Model Drainage Manual for additional design criteria for storage facilities.

892.4 Mixing with Other Waste Streams

Stormwater runoff from State highways will usually be carried to a receiving body of water without being combined with wastewater. Although some combined storm and sanitary sewers do exist, their use should be avoided.

The most common areas of waste stream mixing have been at maintenance stations. These facilities may have combined stormwater and wash rack systems. Because of wash water and rinse water, maintenance stations present unique water quality problems from concentrated levels of pollutant loadings. The preferable design has a separate system for the wash rack so that it is not mixed with stormwater and rinse water. For additional advice on treatment of concentrated waste streams at maintenance stations, contact the Water/Waste Water Unit in the Division of Engineering Services – Structures Design.

Topic 893 – Maintenance Requirements for Stormwater Management Features

893.1 General

As mentioned previously, the ability and the commitment to maintain stormwater management facilities is necessary for their proper operation. The designer must consider the maintenance needs, and the type of maintenance that will take place, in order to provide for adequate access to and within the facility site.

Additionally, the designer should initiate both verbal and written contact with District maintenance to verify the availability of resources to provide proper maintenance and to keep them aware of potential high maintenance items that will be constructed. Initial estimates of how often sediment removal should be performed should be provided by the designer based upon estimated design loadings. Other types of maintenance, such as periodic inspections of embankments, inlet/outlet structures, debris removal, etc. should also be discussed. Due to the large capital investment required for constructing stormwater management facilities, proper maintenance cannot be overlooked.

Refer to the PPDG Section 3.3.4 for a discussion on Maintenance BMPs and the *Maintenance Staff Guide Stormwater Quality Handbook* available on the Maintenance Drainage and Stormwater website.

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