CALIFORNIA
DEPARTMENT OF TRANSPORTATION

manual change transmittal

TITLE DIVISION OF DESIGN HIGHWAY DESIGN MANUAL SIXTH EDITION – CHANGE 03/07/14	APPROVED BY TIMOTHY CRAGGS, Chief	NO. Date Issued: 03/07/14 Page 1 of 4
SUBJECT AREA Table of Contents; List of Figures; List of Tables; Chapters: 60, 80, 200, 400, 500, 800, 810, 830, 850 - 870, 1000; and, Index	ISSUING UNIT DIVISION OF DESIGN	
SUPERCEDES SEE BELOW FOR SPECIFIC PAGE NUMBERS	DISTRIBUTION **ALL HOLDERS OF THE 6TH EDITION, HIGHWAY DESIGN MANUAL**	

The Table of Contents; List of Figures; List of Tables; Chapters: 60, 80, 200, 400, 500, 800, 810, 830, 850 – 870, 1000; and the Index of the Sixth Edition, Highway Design Manual (HDM) have been revised. The changes to the HDM are summarized below with change sheets available on the Department Design website at: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm. Changes include new intersection control guidance in coordination with the recently released Division of Traffic Operations Intersection Control Evaluation (ICE) policy directive. These changes supersede Design Information Bulletin (DIB) Number 80: Roundabouts. Also included are extensive highway drainage design guidance revisions that reflect current nomenclature and correction/update of references to text, tables, figures and website addresses. These changes are effective March 7, 2014, and shall be applied to on-going projects in accordance with HDM Index 82.5 – Effective Date for Implementing Revisions to Design Standards.

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

A summary of the most significant revisions are as follows:

Index 62.4 — **Interchanges and Intersections at Grade, Page 60-3**
Added, updated, and reorganized definitions to be consistent with the changes published in this revision.

Table 82.1B — **Advisory Standards, Page 80-15**
Updated with the new advisory standard included in Index 405.10, Roundabouts.

Topic 201 — **Sight Distance, Page 200-1**
Added the application of stopping sight distance guidance to roundabout design as well as corresponding 10 and 15 mph stopping sight distances in Table 201.1.

Chapter 400 — **Intersections at Grade, Page Various**
Expanded introduction of Chapter 400 to include reference to Traffic Operations Policy Directive (TOPD) Number 13-02: Intersection Control Evaluation (ICE) for direction and procedures on the evaluation, comparison and selection of the intersection types and control strategies identified in Index 401.5, 402.1, and 402.2. Related expanded guidance is provided in new Index 405.10, Roundabouts.

Index 405.10 **Roundabouts, Page 400-34**
New Index 405.10, Roundabouts, supersede Design Information Bulletin Number 80: Roundabouts consistent with updated national research and TOPD Number 13-02. The revised guidance was developed in coordination with the Division of Traffic Operations and the Federal Highway Administration to promote a consistent approach to the selection, design and operation of roundabout intersections.

Figure 405.10 **Roundabout Geometric Elements, Page 400-40**
New Figure including nomenclature associated with roundabout geometric elements.

Index 502.2 **Local Street Interchanges, Page 500-6**
Revised reference to Single Point Interchange Guidelines.

Chapter 800 **Highway Drainage Design, General Aspects, Page Various**
Typographical correction of the term floodplain throughout the Chapter. Revised hydraulic and drainage related reference publications and website addresses.

Index 813.3 **Cross Section, Page 810-4**
Removed reference to revised Index 864.4(3) Water Surface Profiles, Computer Programs, which is no longer applicable.

Index 816.6 **Time of Concentration (Tc) and Travel Time (Tt), Page Various**
Revised Index and Table references.

Topic 819 **Estimating Design Discharge, Page Various**
Revised updated website addresses.

Topic 834 **Roadside Drainage, Page Various**
Revised references to Chapter 860.

Topic 837 **Inlet Design, Page Various**
Revised inlet design guidance where bicycles or pedestrians are anticipated within the roadway or shoulder areas. General update of various inlet types, location and spacing.

Index 851.2 **Selection of Material and Type, Page 850-1**
Updated references to Chapter 860.

Topic 852 **Pipe Materials, Pages Various**
Revised Index references, pipe and pipe lining materials.

Topic 853 **Pipe Liners and Linings for Culvert Rehabilitation, Pages Various**
Updated pipe and pipe lining materials including, but not limited to, those shown in Table 865.2F. Modified plastic pipeliner selection in abrasive conditions per revised Table 853.1B.

Figure 855.1 **Minor Bedload Volume, Page 850-20**
New Figure illustrating an example of one of the lower abrasion levels where minor bedload volumes exist.

Index 855.2	**Abrasion, Pages Various** General update of abrasion level guidance consistent with revised abrasion level definitions, site characteristics, and allowable pipe materials and lining alternative.
Figure 855.2	**Abrasion Test Panels, Page 850-21** Revised Figure number.
Table 855.2A	**Abrasion Levels and Materials, Pages Various** Modified abrasion level definitions.
Table 855.2C	**Guide for Anticipated Service Life Added to Steel Pipe by Abrasive Resistant Protective Coating, Page 850-27** Modified Table to address the removal of polymerized asphalt coating to steel pipes by industry.
Table 855.2D	**Guide for Anticipated Wear to Metal Pipe by Abrasive Channel Materials, Page 850-28** Revised flow velocities per channel materials.
Table 855.2E	**Relative Abrasion Resistance Properties of Pipe and Lining Materials, Page 850-28** Revised relative wear per material.
Table 855.2F	**Guide for Minimum Material Thickness of Abrasive Resistant Invert Protection to Achieve 50 Years of Maintenance-Free Service Life, Page 850-29** Revised abrasion resistance over various pipe and pipe lining materials in various abrasive conditions.
Figure 855.3A	**Minimum Thickness of Metal Pipe for 50-Year Maintenance-Free Service Life, Page 850-32** Revised Index references in Notes.
Table 855.4A	**Guide for the Protection of Cast-In-Place and Precast Reinforced and Unreinforced Concrete Structures Against Acid and Sulfate Exposure Conditions, Page 850-35** Revised sulfate concentrations.
Table 855.4B	**Guide for Minimum Cover Requirements for Cast-In-Place and Precast Reinforced Concrete Structures for 50-Year Design Life in Chloride Environments, Page 850-36** Revised references to Standard Specifications.
Table 856.3A	**Corrugated Steel Pipe Helical Corrugations, Page 850-39** Revised metal thicknesses for various diameter corrugated steel pipe.
Table 856.3D	**Corrugated Steel Pipe Arches 2⅔" x ½"Helical or Annular Corrugations, Page 850-42** Revised Note.

Table 856.3J — **Corrugated Aluminum Pipe Arches 2⅔" x ½"Helical or Annular Corrugations, Page 850-48**
Deleted previous Note (2) regarding use under abrasive conditions.

Table 856.3P — **Structural Aluminum Plate Pipe Arches 9" x 2½" Corrugations, Page 850-54**
Deleted previous Note (1) regarding use under abrasive conditions.

Table 856.4 — **Thermoplastic Pipe Fill Height Tables, Page 850-55**
Revised title of last table to Polyvinyl Chloride (PVC) Corrugated Pipe with Smooth Interior.

Table 856.5 — **Minimum Thickness of Cover for Culverts, Page 850-56**
Revised minimum thickness of cover over aluminum spiral rib pipe, S less than or equal to 48 inches, and plastic pipes.

Index 857.3 — **Alternative Pipe Culvert (APC) and Pipe Arch Culvert List, Page 850-59**
Revised reference title to the Plans Preparation Manual.

Chapter 860 — **Roadside Channels, Pages Various**
Complete update and rewrite to reflect a change in guidance philosophy from velocity based to shear stress based analysis of channel erosion. Chapter 860 focuses on smaller manmade channels, which will accommodate a future change in Chapter 870 to natural channels.

Chapter 870 — **Channel and Shore Protection – Erosion Control, Pages Various**
Typographical correction of the term floodplain throughout the Chapter. The revision of Chapter 860 eliminated a Table in Chapter 870.

Index 873.3 — **Armor Protection, Page 870-40**
Revised references to Chapter 860.

Index 1003.5 — **Miscellaneous Criteria, Page 1000-14**
Revised reference to updated inlet design guidance when considering grate inlets within the roadway or shoulder areas.

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

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and expressways apply to controlled access highways.

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

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

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

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

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

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

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

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

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

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

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

(7) *Street or Road.*

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

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

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

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

(e) Private Road or Driveway—A way or place in private ownership and used for travel by the owner and those having express or implied permission from the owner, but not by other members of the public.

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

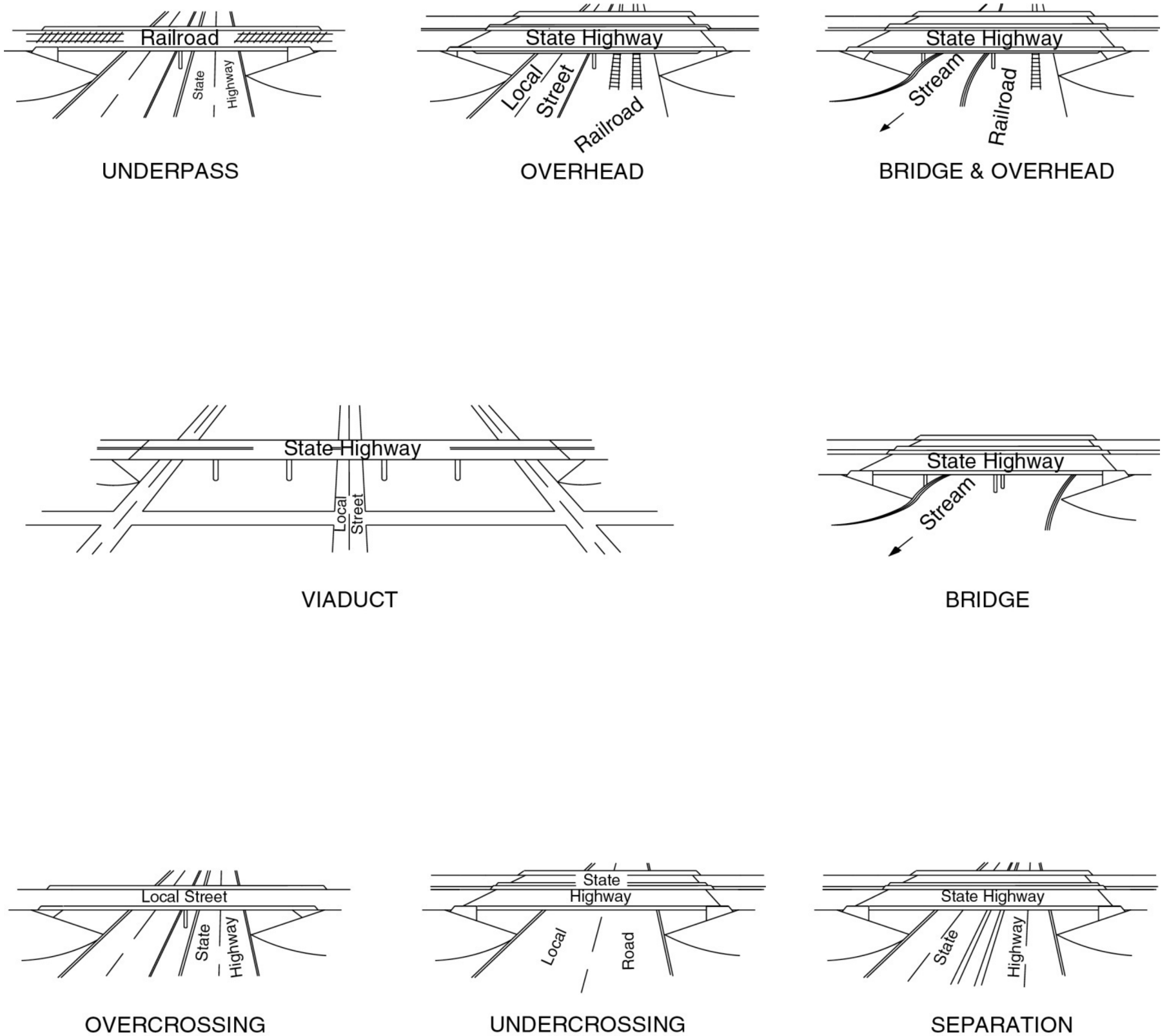
(g) Toll Road, Bridge or Tunnel--A highway, bridge, or tunnel open to traffic only upon payment of a toll or fee.

62.4 Interchanges and Intersections at Grade

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

(2) *Circulatory Roadway.* The curved roadbed that users of a roundabout travel on in a counterclockwise direction around the central island.

Figure 62.2
Types of Structures



(3) *Channelization.* The separation or regulation of conflicting movements into definite paths of travel by the use of pavement markings, raised islands, or other suitable means to facilitate the safe and orderly movement of vehicles, bicycles and pedestrians.

(4) *Crosswalk.* Crosswalk is either:

(a) That portion of a roadway included within the prolongation or connection of the boundary lines of sidewalks at intersections where the intersecting roadways meet at approximately right angles, except the prolongation of such lines from an alley across a street.

(b) Any portion of a roadway distinctly indicated for pedestrian crossing by lines or other markings on the surface.

(5) *Geometric Design.* The arrangement of the visible elements of a road, such as alignment, grades, sight distances, widths, slopes, etc.

(6) *Gore.* The area immediately beyond the divergence of two roadbeds bounded by the edges of those roadbeds.

(7) *Grade Separation.* A crossing of two highways, highway and local road, or a highway and a railroad at different levels.

(8) *Inscribed Circle Diameter.* The distance across the circle of a roundabout, inscribed by the outer curb (or edge) of the circulatory roadway. It is the sum of the central island diameter and twice the circulatory roadway width.

(9) *Interchange.* A system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of vehicles between two or more roadways on different levels.

(10) *Interchange Elements.*

(a) Branch Connection--A multilane connection between two freeways.

(b) Freeway-to-freeway Connection--A single or multilane connection between freeways or any two high speed facilities.

(c) Ramp--A connecting roadway between a freeway or expressway and another highway, road, or roadside area.

(11) *Intersection.* The general area where two or more roadways join or cross, including the roadway and roadside facilities for movements in that area.

(12) *Island.* A defined area between roadway lanes for control of vehicle movements or for pedestrian refuge. Within an intersection a median or an outer separation is considered an island.

(13) *Landscape Buffer/Strip.* A planted section adjacent to the legs of a roundabout that separates users of the roadway from users of the shared use/Class I Bikeway and assists with guiding pedestrians to the designated crossing locations. Also known as “way finding.”

(14) *Minimum Turning Radius.* The radius of the path of the outer front wheel of a vehicle making its sharpest turn.

(15) *Offset Left-Turn Lanes.* Left-turn lanes are shifted as far to the left as practical rather than aligning the left-turn lane exactly parallel with and adjacent to the through lane.

(16) *Offtracking.* The difference between the paths of the front and rear wheels of a vehicle as it negotiates a turn.

(17) *Pedestrian Refuge.* A section of pavement or sidewalk, completely surrounded by asphalt or other road materials, where users can stop before completing the crossing of a road.

(18) *Roundabout.* A type of circular intersection with specific geometric and traffic control features that in combination lower speed operations and lower speed differentials among all users immediately prior to, through, and beyond the intersection. Vehicle speed is controlled by deflection in the path of travel, and the “yield upon entry” rule for traffic approaching the roundabout’s circulatory roadway. Curves and deflections are introduced that limit operating speeds.

(19) *Splitter Island.* A raised or painted traffic island that separates traffic in opposing

directions of travel. They are typically used at roundabouts and on the minor road approaches to an intersection.

(20) *Skew Angle.* The complement of the acute angle between two centerlines which cross.

(21) *Swept width.* The total width needed by the vehicle body to traverse a curve; it is the distance measured along the curve radius from the outer front corner of the body to the inner rear corner of the body as the vehicle traverses around a curve. This width is used to determine clearance to objects, such as signs, poles, etc., as well as vehicles, bicycles, and pedestrians.

(22) *Tracking width.* The total width needed by the tires to traverse a curve; it is the distance measured along the curve radius from the outer front tire track to the inner rear tire track as the vehicle traverses around a curve. This width is used to determine the minimum width required for the vehicle turning. Consideration for additional width may be needed for other vehicles, bicycles and pedestrians.

(23) *Truck Apron.* The traversable portion of the roundabout central island adjacent to the circulatory roadway that may be needed to accommodate the wheel tracking of large vehicles. A truck apron is sometimes provided on the outside of the circulatory roadway, but cannot encroach upon the pedestrian crossing.

(24) *Weaving Section.* A length of roadway, designed to accommodate two traffic streams merging and dividing within a short distance.

(25) *Wheelbase.* For single-unit vehicles, the distance from the first axle to the single rear axle or, in the case of a tandem or triple set of rear axles, to the center of the group of rear axles. See Topic 404

62.5 Landscape Architecture

(1) *"A" Soil Horizon.* Formed below the "O" soil horizon layer, defined in part (9) below, where mineral matter is mixed with decayed organic matter.

(2) *Classified Landscaped Freeway.* A classified landscaped freeway is a planted section of freeway that meets the criteria established by the California Code of Regulations Outdoor Advertising Regulations, Title 4, Division 6. This designation is used in the control and regulation of outdoor advertising displays.

(3) *Duff*. A vegetative material that has been collected and removed from the project during clearing and grubbing activities, chipped or ground up and stockpiled for reapplication to the final slope surface.

(4) *Highway Planting.* Highway planting addresses safety requirements, provides compliance with environmental commitments, and assists in the visual integration of the transportation facility within the existing natural and built environment. Highway planting provides planting to satisfy legal mandates, environmental mitigation requirements, Memoranda of Understanding or Agreement between the Department and local agencies for aesthetics or erosion control. Highway planting also includes roadside management strategies that improve traveler and worker safety by reducing the frequency and duration of maintenance worker exposure.

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

Highway planting, funded and maintained by the Department on conventional highways, is limited to planting that provides: safety improvements, erosion control/storm water pollution prevention, revegetation, and required mitigation planting. Highway planting on freeways, controlled access highways and expressways, funded and maintained by the Department, is limited to areas that meet specific criteria. See Chapter 29 "Landscape Architecture" of the Project Development Procedures Manual (PDPM) for more detailed information regarding warranted planting.

(5) *Highway Planting Restoration.* Highway planting restoration provides for replacement, restoration, and rehabilitation of existing vegetation damaged by weather, acts of nature or deterioration, to integrate the facility with

the adjacent community and surrounding environment. Highway planting restoration also provides erosion control to comply with National Pollutant Discharge Elimination System (NPDES) permit requirements. These projects include strategies designed to protect the safety of motorists and maintenance workers by minimizing recurrent maintenance activities.

(6) *Highway Planting Revegetation.* Highway planting revegetation provides planting as mitigation for native vegetation damaged or removed due to a roadway construction project. Highway planting revegetation may include irrigation systems as appropriate. Highway planting revegetation, required due to the impacts of a roadway construction project, must be programmed and funded by the parent roadway project.

(7) *Imported Topsoil*. Soil that is delivered onto a project from a commercial source and is fertile, friable soil of loamy character that contains organic matter.

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

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

(10) *Park and Ride.* A paved area for parking which provides a connection point for public access to a variety of modal options. See Topic 905.

(11) *Replacement Highway Planting.* Replacement highway planting replaces vegetation installed by the Department or others, that has been damaged or removed due to transportation project construction. Replacement highway planting may also include irrigation modifications and/or replacement. Replacement highway planting required due to the impacts of a roadway construction project must be programmed in conjunction with and funded from the parent roadway project.

(12) *Required Mitigation Planting.* Required mitigation planting provides planting and other work necessary to mitigate environmental impacts due to roadway construction. The word "required" indicates that the work is necessary to meet legally required environmental mitigation or permit requirements. Required mitigation planting may be performed within the operational right of way, immediately adjacent to the highway or at an offsite location as determined by the permit. A planting project for required mitigation due to the impacts of a roadway construction project must be programmed and funded by the parent roadway project.

(13) *Safety Roadside Rest Area System*. The safety roadside rest area system is a component of the highway system providing roadside areas where travelers can stop, rest and manage their travel needs. Planned with consideration of alternative stopping opportunities such as truck stops, commercial services, and vista points, the rest area system provides public stopping opportunities where they are most needed, usually between large towns and at entrances to major metropolitan areas. Within the safety roadside rest system, individual rest areas may include vehicle parking, picnic tables, sanitary facilities, telephones, water, tourist information panels, traveler service information facilities and vending machines. See Topic 903.

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

(15) *Vista Point.* Typically a paved dedicated area beyond the shoulder that permits travelers to stop and view a scenic area. In addition to parking areas, amenities such as trash receptacles, interpretive displays, and in some cases, rest rooms, drinking water and telephones may be provided. See Topic 904.

62.6 Right of Way

(1) *Acquisition.* The process of obtaining right of way.

(2) *Air Rights.* The property rights for the control or specific use of a designated airspace involving a highway.

(3) *Appraisal.* An expert opinion of the market value of property including damages and special benefits, if any, as of a specified date, resulting from an analysis of facts.

(4) *Business District (or Central Business District).* The commercial and often the geographic heart of a city, which may be referred to as “downtown.” Usually contains retail stores, theatres, entertainment and convention venues, government buildings, and little or no industry because of the high value of land. Historic sections may be referred to as “old town.”

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

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

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

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

(9) *Encroachment.* Any structure, object, or activity of any kind or character which is within the right of way, but not a part of the State facility, or serving a transportation need.

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

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

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

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

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

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

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

62.7 Pavement

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

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

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

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

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

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

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

(7) *California R-Value.* A measure of resistance to deformation of the soils under saturated conditions and traffic loading as determined

by the stabilometer test (CT301). The California R-value, also referred to as R-value, measures the supporting strength of the subgrade and subsequent layers used in the pavement structure. For additional information, see Topic 614.

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

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

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

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

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

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

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

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

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

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

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

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

(20) *Equivalent Single Axle Loads (ESAL's).* The number of 18-kip standard single axle load repetitions that would have the same damage effect to the pavement as an axle of a specified magnitude and configuration. See Index 613.3 for additional information.

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

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

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

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

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

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

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

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

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

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

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

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

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

(34) *Pavement Design Life.* Also referred to as performance period, pavement design life is the period of time that a newly constructed or rehabilitated pavement is engineered to perform before reaching a condition that requires CAPM, (see Index 603.4). The selected pavement design life varies depending on the characteristics of the highway facility, the objective of the project, and projected vehicle volume and loading.

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

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

(37) *Pavement Service Life.* Is the actual period of time that a newly constructed or rehabilitated pavement structure performs satisfactorily before reaching its terminal serviceability or a condition that requires major rehabilitation or reconstruction. Because of the many independent variables involved, pavement

service life may be considerably longer or shorter than the design life of the pavement. For further discussion, see Topic 612.

(38) *Pavement Structure.* See Pavement.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

(53) *Subgrade.* Also referred to as basement soil, is that portion of the roadbed consisting of

native or treated soil on which pavement surface course, base, subbase, or a layer of any other material is placed.

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

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

62.8 Highway Operations

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

(2) *Delay*. The time lost while road users are impeded by some element over which the user has no control.

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

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

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

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

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

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

(9) *Managed Lanes.* Lanes that are proactively managed in response to changing operating conditions in efforts to achieve improved efficiency and performance. Typically employed on highways with increasing recurrent traffic congestion and limited resources.

(a) High-Occupancy Vehicle (HOV) Lanes--An exclusive lane for vehicles carrying the posted number of minimum occupants or carpools, either part time or full time.

(b) High Occupancy Toll (HOT) Lanes--An HOV lane that allows vehicles qualified as carpools to use the facility without a fee, while vehicles containing less than the required number of occupants to pay a toll. Tolls may change based on real time conditions (dynamic) or according to a schedule (static).

(c) Express Toll Lanes--Facilities in which all users are required to pay a toll, although HOVs may be offered a discount. Tolls may be dynamic or static.

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

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

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

(13) *Speed.*

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

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

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

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

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

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

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

(15) Traffic Control Devices.

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

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

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

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

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

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

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

62.9 Drainage

See Chapter 800 for definition of drainage terms.

62.10 Users

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

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

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

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

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

and typically run within their own right of way corridor.

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

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

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

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

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

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

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

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

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

CHAPTER 200 GEOMETRIC DESIGN AND STRUCTURE STANDARDS

Topic 201 - Sight Distance

Index 201.1 - General

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

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

See Chapter 1000 for Class I bikeway sight distance guidance.

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

201.2 Passing Sight Distance

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

**Table 201.1
Sight Distance Standards**

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

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

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

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

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

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

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

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

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

201.3 Stopping Sight Distance

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

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

201.4 Stopping Sight Distance at Grade Crests

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

201.5 Stopping Sight Distance at Grade Sags

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

If headlight sight distance is not obtainable at grade sags, lighting may be considered. The Design Coordinator and the HQ Traffic Liaison shall be contacted to review proposed grade sag lighting to determine if such use is appropriate.

201.6 Stopping Sight Distance on Horizontal Curves

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

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

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

CHAPTER 400 INTERSECTIONS AT GRADE

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

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

Topic 401 - Factors Affecting Design

Index 401.1 - General

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

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

401.2 Human Factors

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

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

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

(3) *The Pedestrian.* Understanding how pedestrians will use an intersection is critical because pedestrian volumes, their age ranges, physical ability, etc. all factor in to their startup time and the time it takes them to cross an intersection and thus, dictates how to design

the intersection to avoid potential conflicts with bicyclists and motor vehicles. The guidance in this chapter specifically relates to pedestrian travel within intersections on the State highway system. See Topic 105, Pedestrian Facilities, Design Information Bulletin 82 - "Pedestrian Accessibility Guidelines for Highway Projects," the AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities, and the California Manual on Uniform Traffic Control Devices (California MUTCD) for additional guidance.

401.3 Traffic Considerations

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

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

Table 401.3 compares vehicle characteristics to intersection design elements.

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

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

401.4 The Physical Environment

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

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

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

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

Table 401.3

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

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

401.5 Intersection Type

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

(1) *Geometric Configurations*

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

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

(c) Alternative Intersection Designs – various effective geometric alternatives to traditional designs that can reduce crashes and their severity, improve operations, reduce congestion and delay typically by reducing or altering the number of conflict points; these alternatives include geometric design features such as intersections with displaced left-turns or variations on U-turns.

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

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

(b) All-Way Stop Control

(c) Signal Control

(d) Yield Control (Roundabout)

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

401.6 Transit

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

Topic 402 - Operational Features Affecting Design

402.1 Capacity

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

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

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

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

402.2 Collisions

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

The type of traffic control affects the type of collisions. Signalized intersections tend to have more rear end and same-direction sideswipes than intersections with "STOP"-control on minor legs. Roundabouts experience few angle or crossing collisions. Roundabouts reduce the frequency and severity of collisions, especially when compared to the performance of signalized intersections in high speed environments. Other alternative intersection types are configurations to consider for minimizing the number of conflict points.

(2) *Undesirable Geometric Features.*

- Inadequate approach sight distance.
- Inadequate corner sight distance.
- Steep grades.
- Five or more approaches.
- Presence of curves within intersections (unless at roundabouts).
- Inappropriately large curb radii.
- Long pedestrian crossing distances.
- Intersection Angle <75 degrees (see Topic 403).

402.3 On-Street Parking

On-street parking generally decreases through-traffic capacity, impedes traffic flow, and increases crash potential. Where the primary service of the arterial is the movement of vehicles, it may be desirable to prohibit on-street parking on State highways in urban and suburban expressways and rural arterial sections. However, within urban and suburban areas and in rural communities located on State highways, on-street parking should be considered in order to accommodate existing land uses. Where adequate off-street parking facilities are not available, the designer should consider on-street parking, so that the proposed highway improvement will be compatible with the land use. On-street parking as well as off-street parking needs to comply with DIB 82. See AASHTO, A Policy on Geometric Design of Highways and Streets for additional guidance related to on-street parking.

402.4 Consider All Users

Intersections should accommodate all users of the facility, including vehicles, bicyclists, pedestrians and transit. Bicycles have all the rights and responsibilities as motorist per the California Vehicle Code, but should have separate consideration of their needs, even separate facilities if volumes warrant. Pedestrians should not be prohibited from crossing one or more legs of an intersection, unless no other safe alternative exists. Pedestrians can be prohibited from crossing one or more legs of an intersection if a reasonable alternate route exists and there is a demonstrated need to do so. All pedestrian facilities shall be ADA compliant as outlined in DIB 82. Transit needs should be determined early in the planning and design phase as their needs can have a large impact on the performance of an intersection. Transit stops in the vicinity of intersections should be evaluated for their effect on the safety and operation of the intersection(s) under study. See Index 205.6 for additional information.

402.5 Speed-Change Areas

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

Topic 403 - Principles of Channelization

403.1 Preference to Major Movements

The provision of direct free-flowing high-standard alignment to give preference to major movements is good channelization practice. This may require some degree of control of the minor movements such as stopping, funneling, or even eliminating them. These controlling measures should conform to natural paths of movement and should be introduced gradually to promote smooth and efficient operation.

403.2 Areas of Conflict

Large multilane undivided intersection areas are undesirable. The hazards of conflicting movements are magnified when motorists, bicyclists, and pedestrians are unable to anticipate movements of other users within these areas. Channelization reduces areas of conflict by separating or regulating traffic movements into definite paths of travel by the use of pavement markings or traffic islands.

Multilane undivided intersections, even with signalization, are more difficult for pedestrians to cross. Providing pedestrian refuge islands enable pedestrians to cross fewer lanes at a time.

See Index 403.7 for traffic island guidance when used as pedestrian refuge. Curb extensions shorten crossing distance and increase visibility. See Index 303.4 for curb extensions.

403.3 Angle of Intersection

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

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

- The shortest crossing distance for motor vehicles, bicycles, and pedestrians.
- Sight lines which optimize corner sight distance and the ability of motorists to judge the relative position and speed of approach traffic.
- Intersection geometry that can reduce vehicle turning speeds so collisions are more easily avoided and the severity of collisions are minimized.
- Intersection geometry that sends a message to turning bicyclists and motorists that they are making a turning movement and should yield as appropriate to through traffic on the roadway they are leaving, to traffic on the receiving roadway, and to pedestrians crossing the intersection.

Minor deviations from right angles are generally acceptable provided that the potentially detrimental impact on visibility and turning movements for large trucks (see Topic 404) can be mitigated. However, large deviations from right angles may decrease visibility, hamper certain turning operations, and will increase the size of the intersection and therefore crossing distances for bicyclists and pedestrians, may encourage high speed turns, and may reduce yielding by turning traffic. <u>When a right angle cannot be provided due to physical constraints, the interior angle should be designed as close to 90 degrees as is practical, but should not be less than 75 degrees. Mitigation should be considered for the affected intersection design features.</u> (See Figure 403.3A). A 75 degree angle does not unreasonably increase the crossing distance or generally decrease visibility. Class II bikeway crossings at railroads follow similar guidance to Class I bikeway crossings at railroads, see Index 1003.5(3), and Figure 403.3B.

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

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

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

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

403.4 Points of Conflict

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

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

See AASHTO, A Policy on Geometric Design of Highways and Streets for additional guidance on speed-change lanes.

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

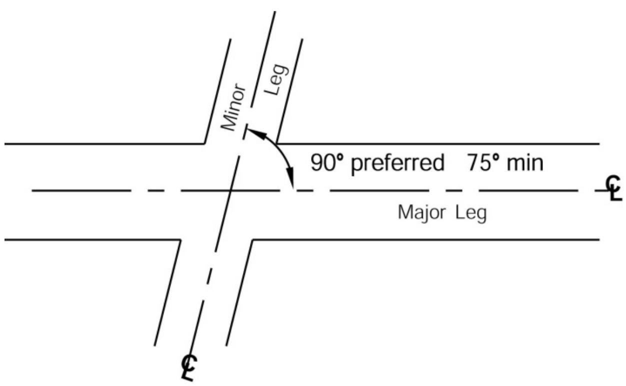
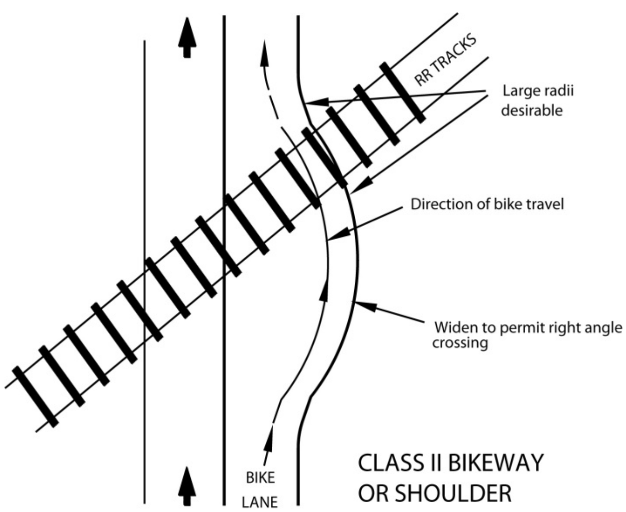


Figure 403.3.B
Class II Bikeway
Crossing Railroad



403.5 (Currently Not In Use)

403.6 Turning Traffic

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

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

(1) Treatment of Intersections with Right-Turn-Only Lanes. Most motor vehicle/bicycle collisions occur at intersections. For this reason, intersection design should be accomplished in a manner that will minimize confusion by motorists and bicyclists, eliminate ambiguity and induce all road users to operate in accordance with the statutory rules of the road in the California Vehicle Code. Right-turn-only lanes should be designed to meet user expectations and reduce conflicts between vehicles and bicyclists.

Figure 403.6A illustrates a typical at-grade intersection of multilane streets without right-turn-only lanes. Bike lanes or shoulders are included on all approaches. Some common movements of motor vehicles and bicycles are shown. A prevalent crash type is between straight-through bicyclists and right-turning motorists, who do not yield to through bicyclists.

<u>Optional right-turn lanes should not be used in combination with right-turn-only lanes on roads where bicycle travel is permitted.</u> The use of optional right-turn lanes in combination with right-turn-only lanes is not recommended in any case where a Class II bike lane is present. This may increase the need for dual or triple right-turn-only lanes, which have

challenges with visibility between turning vehicles and pedestrians. Multiple right-turn-only lanes should not be free right-turns when there is a pedestrian crossing. If there is a pedestrian crossing on the receiving leg of multiple right-turn-only lanes, the intersection should be controlled by a pedestrian signal head, or geometrically designed such that pedestrians cross only one turning lane at a time.

Locations with right-turn-only lanes should provide a minimum 4-foot width for bicycle use between the right-turn and through lane when bikes are permitted. Configurations that create a weaving area without defined lanes should not be used.

Signing and delineation for bicycle lanes at intersections is shown in California MUTCD.

Figure 403.6B depicts an intersection with a left-turn-only bicycle lane, which should be considered when bicycle left-turns are common. A left-turn-only bicycle lane may be considered at any intersection and should always be considered as a tool to provide mobility for bicyclists. Signing and delineation options for bicycle left-turn-only lanes are shown in California MUTCD.

(2) *Intersections at Interchange Design.* The design of at-grade intersections at interchanges should be accomplished in a manner that will minimize confusion of motorists and bicyclists. High speed, uncontrolled, low angle entries and exits from freeway ramps should not be used at the intersection of the ramps with the local road. These types of ramp intersections are appropriate for ramp merges onto freeways, but not at ramp intersections with local roads. Higher angle intersections tend to reduce speeds at conflict points between motorists, bicyclists, and pedestrian.

403.7 Refuge Areas

Traffic islands should be used to provide refuge areas for bicyclists and pedestrians. See Index 405.4 for further guidance.

403.8 Prohibited Turns

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

403.9 Effective Signal Control

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

The California MUTCD has warrants for the placement of signals to control vehicular, bicycle and pedestrian traffic. Pedestrian activated signals must be provided at pedestrian street crossings at multi-lane channelized turn lanes at roundabouts and other signalized intersections. Consideration of a leading pedestrian interval where the potential for conflicts are high should be made. Signal phasing to accommodate pedestrians may have a dramatic effect on intersection capacity. See Topic 406, the Highway Capacity Manual and the California MUTCD for additional information.

403.10 Installation of Traffic Control Devices

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

403.11 Summary

- Give preference to the major move(s).
- Reduce areas of conflict.
- Reduce the duration of conflicts.
- Cross traffic at right angles or skew no more than 75 degrees. (90 degrees preferred.)
- Separate points of conflict.
- Provide speed-change areas and separate turning lanes where appropriate.
- Provide adequate width to shadow turning traffic.

- Restrict undesirable moves with traffic islands.
- Coordinate channelization with effective signal control.
- Install signs in traffic islands when necessary but avoid building conflicts one or more modes of travel.
- Consider all users.

403.12 Other Considerations

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

Topic 404 - Design Vehicles

404.1 General

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

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

404.2 Design Considerations

It may not be necessary to provide for design vehicle turning movements at all intersections along the State route if the design vehicle's route is restricted or it is not expected to use the cross street frequently. Discuss with Traffic Operation and the local agency before a turning movement is not provided. The goal is to minimize as much as possible conflicts between vehicles, bicycles, pedestrians, and other users of the street, while providing the minimum curb radii appropriate for the given situation. The designer may reference the AASHTO Green Book to select the design vehicle to analyze turning movements to and from the State route. However, turning movements of the State route design vehicle should also be analyzed to determine the impacts from their occasional use.

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

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

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

(1) *Traveled way.*

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

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

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

- In urban, city or town centers (rural main streets) with posted speeds less than 40 miles per hour in severely constrained situations, if truck or bus use is low, consideration may be given to reducing the right-turn lane width to 10 feet with approval of a design exception.
- Shoulder widths may also be considered for reduction under constricted situations. Whenever possible, at least a 2-foot offset should be provided where the right-turn lane is adjacent to a curb. Entire omission of the shoulder should only be considered in the most severely constricted situations and where an 11-foot lane can be constructed. Gutter pans can be included within a shoulder, but cannot be included as part of the lane width.

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

(b) Tapers--Approach tapers are usually unnecessary since main line traffic need not be shifted laterally to provide space for the right-turn lane. If, in some rare instances, a lateral shift were needed, the approach taper would use the same formula as for a left-turn lane.

Bay tapers are treated as a mirror image of the left-turn bay taper.

(c) Deceleration Lane Length--The conditions and principles of left-turn lane deceleration apply to right-turn deceleration. Where full deceleration is desired off the high-speed through lanes, the lengths in Table 405.2B should be used. Where partial deceleration is permitted on the through lanes because of limited right of way or other constraints, average running speeds in Table 405.2B may be reduced 10 miles per hour to 20 miles per hour for a lower entry speed. For example, if the main line speed is 50 miles per hour and a 10 miles per hour deceleration is permitted on the through lanes, the deceleration length may be that required for 40 miles per hour.

(d) Storage Length--Right-turn storage length is determined in the same manner as left-turn storage length. See Index 405.2(2)(e).

(3) Right-turn Lanes at Off-ramp Intersections. Diamond off-ramps with a free right-turn at the local street and separate right-turn off-ramps around the outside of a loop will likely cause conflict as traffic volumes increase. Serious conflicts occur when the right-turning vehicle must weave across multiple lanes on the local street in order to turn left at a major cross street close to the ramp terminal. Furthermore, free right-turns create sight distance issues for pedestrians and bicyclists crossing the off-ramp, or pedestrians crossing the local road. Also, rear-end collisions can occur as right-turning drivers slow down or stop waiting for a gap in local street traffic. Free right-turns usually end up with ”YIELD”, ”STOP”, or signal controls thus defeating their purpose of increasing intersection capacity.

405.4 Traffic Islands

A traffic island is an area between traffic lanes for channelization of bicycle and vehicle movements or for pedestrian refuge. An island may be defined by paint, raised pavement markers, curbs, pavement edge, or other devices. The California MUTCD should be referenced when considering the placement of traffic islands at signalized and unsignalized locations. For splitter island guidance at roundabouts, see Index 405.10(13).

Traffic islands usually serve more than one function. These functions may be:

(a) Channelization to confine specific traffic movements into definite channels;

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

(c) Refuge, to aid users crossing the roadway.

Generally, islands should present the least potential conflict to approaching or crossing bicycles and vehicles, and yet perform their intended function.

(1) Design of Traffic Islands. Island sizes and shapes vary from one intersection to another. They should be large enough to command

attention. Channelizing islands should not be less than 50 square feet in area, preferably 75 square feet. Curbed, elongated divisional median islands should not be less than 4 feet wide and 20 feet long. All traffic islands placed in the path of a pedestrian crossing must comply with DIB 82. See the Standard Plans for typical island passageway details.

The approach end of each island should be offset 3 feet to the left and 5 feet to the right of approaching traffic, using standard 1:15 parabolic flares, and clearly delineated so that it does not surprise the motorist or bicyclist. These offsets are in addition to the shoulder widths shown in Table 302.1. Table 405.4 gives standard parabolic flares to be used in island design. On curved alignment, parabolic flares may be omitted for small triangular traffic islands whose sides are less than 25 feet long.

The approach nose of a divisional island should be highly visible day and night with appropriate use of signs (reflectorized or illuminated) and object markers. The approach nose should be offset 3 feet from the through traffic to minimize accidental impacts.

(2) *Delineation of Traffic Islands.* Generally, islands should present the least potential conflict to approaching traffic and yet perform their intended function. See Index 303.2 for appropriate curb type. Islands may be designated as follows:

(a) Raised paved areas outlined by curbs.

(b) Flush paved areas outlined by pavement markings.

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

On facilities with posted speeds over 40 miles per hour, the use of any type of curb is discouraged. Where curbs are to be used, they should be located at or outside of the shoulder edge, as discussed in Index 303.5.

In rural areas, painted channelization supplemented with raised pavement markers may be more appropriate than a raised curbed channelization. This design is as forgiving as possible and decreases the consequence of a driver's or bicyclist's failure to detect or recognize the curbed island. Consideration for snow removal operations should be determined where appropriate.

In urban areas, posted speeds less than or equal to 40 miles per hour allow more frequent use of curbed islands. Local agency requirements and matching existing conditions are factors to consider.

(3) *Pedestrian Refuge*

Pedestrian refuge islands allow pedestrians to cross fewer lanes at a time while judging conflicts separately. They also provide a refuge so slower pedestrians can wait for a gap in traffic. <u>Traffic islands used as pedestrian refuge should be large enough to provide a minimum of 6 feet in the direction of pedestrian travel</u>. All traffic islands placed in the path of a pedestrian crossing must be accessible, refer to DIB 82 and the Standard Plans for further guidance. An example of a traffic island that serves as a pedestrian refuge is shown on Figure 405.4.

405.5 Median Openings

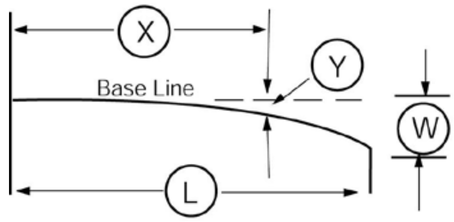
(1) *General.* Median openings, sometimes called crossovers, provide for crossings of the median at designated locations. Except for emergency passageways in a median barrier, median openings are not allowed on urban freeways.

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

(2) *Spacing and Location.* By a combination of interchange ramps and emergency passageways, provisions for access to the opposite side of a freeway may be provided for law enforcement, emergency, and maintenance vehicles to avoid extreme out-of-direction travel. Access should not be more frequent than at three-mile intervals. See Chapter 7 of

Table 405.4

Parabolic Curb Flares Commonly Used



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

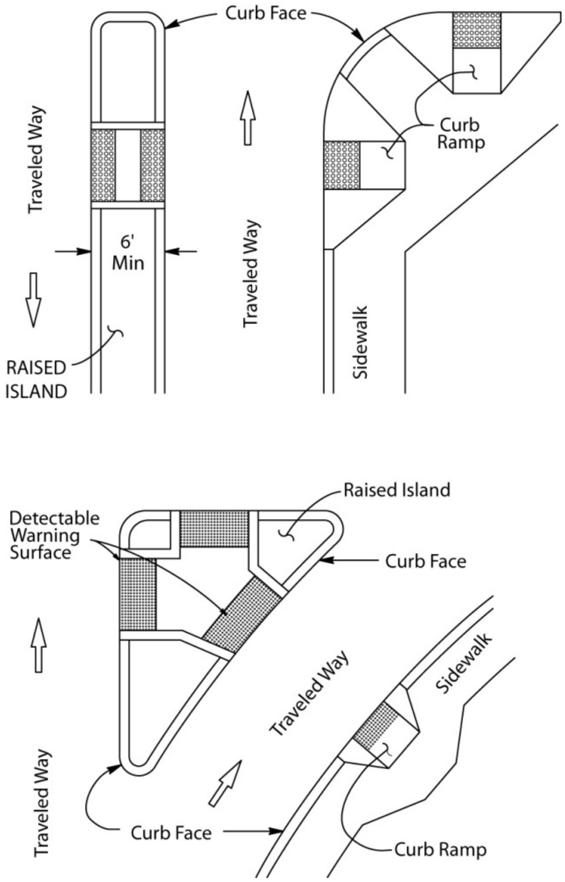
L = Length of flare in feet
W = Maximum offset in feet
X = Distance along base line in feet
Y = Offset from base line in feet

W is shown in table thus

OFFSET IN FEET FOR GIVEN "X" DISTANCE																
Distance X / L Length of Flare	10	15	20	25	30	40	45	50	60	70	75	80	90	100	110	120
1:5 FLARES																
25	0.80	1.80	3.20	5.00												
50	0.40		1.60		3.60	6.40		10.00								
1:10 FLARES																
50	0.20		0.80		1.80	3.20		5.00								
100	0.10		0.40		0.90	1.60		2.50	3.60	4.90		6.40	8.10	10.00		
1:15 FLARES																
45	0.15		0.59		1.33	2.37	3.00									
75	0.09		0.36		0.80	1.42		2.22	3.20	4.36	5.00					
90	0.07		0.30		0.67	1.19		1.85	2.67	3.63		4.74	6.00			
120	0.06		0.22		0.50	0.89		1.39	2.00	2.72		3.56	4.50	5.56	6.72	8.00

Figure 405.4

Pedestrian Refuge Island



the Traffic Manual for additional information on the design of emergency passageways.

Emergency passageways should be located only where decision sight distance is available (see Table 201.7).

Median openings at close intervals on other types of highways create conflicts with high speed through traffic. Median openings should be spaced at intervals no closer than 1600 feet. If a median opening falls within 300 feet of an access opening, it should be placed opposite the access opening.

(3) *Length of Median Opening.* For any three or four-leg intersection on a divided highway, the length of the median opening should be at least as great as the width of the crossroads pavement, median width, and shoulders. An important factor in designing median openings is the path of the design vehicle making a minimum left turn at 5 miles per hour to 10 miles per hour. The length of median opening varies with width of median and angle of intersecting road.

Usually a median opening of 60 feet is adequate for 90 degree intersections with median widths of 22 feet or greater. When the median width is less than 22 feet, a median opening of 70 feet is needed. When the intersection angle is other than 90 degrees, the length of median opening should be established by using truck turn templates (see Index 404.3).

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

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

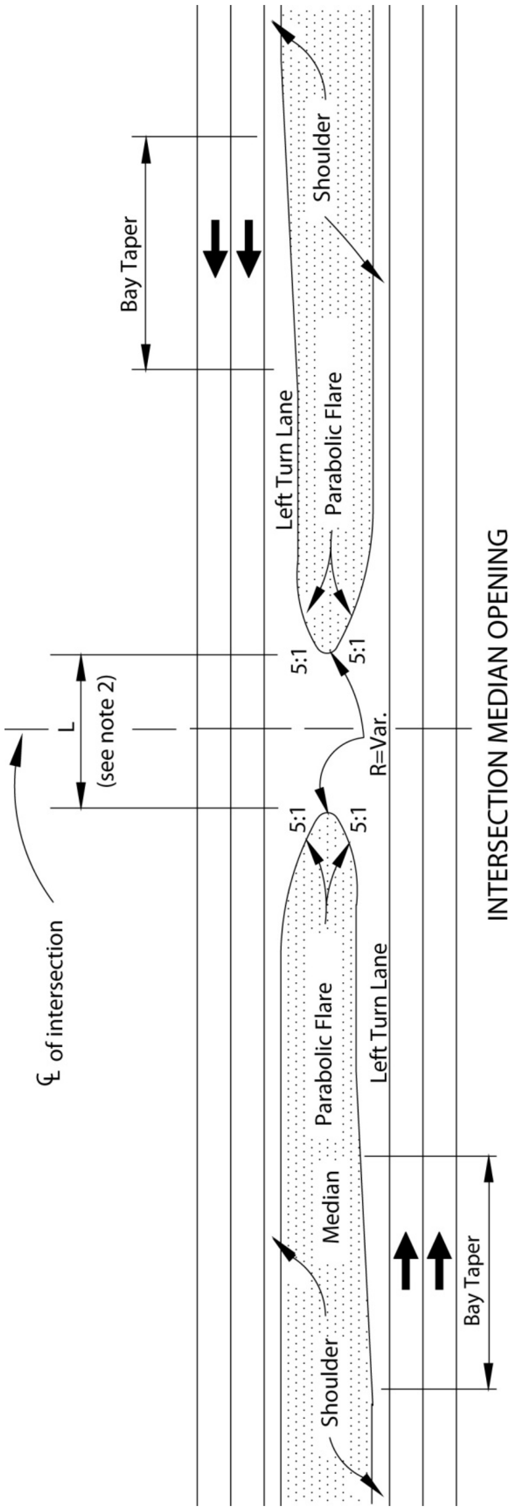
405.6 Access Control

The basic guidance which govern the extent to which access rights are to be acquired at interchanges (see Topic 104, Index 205.1 and 504.8 and the PDPM) also apply to intersections at grade on expressways. Cases of access control which frequently occur at intersections are shown in Figure 405.7. This illustration does not presume to cover all situations. Where required by traffic conditions, access should be extended in order to ensure proper operation of the expressway lanes. Reasonable variations which observe the basic principles referred to above are acceptable.

However, negative impacts on the mobility needs of pedestrians, bicyclists, equestrians, and transit users need to be assessed. Pedestrians and bicyclists are sensitive to additional out of direction travel.

Figure 405.5

Typical Design for Median Openings



NOTES:

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

② L = Length of median opening: varies with width of median and angle of intersecting road. Usually for 90° intersection, L = 60 feet for median of 22 feet and wider. L = 70 feet for medians narrower than 22 feet.

③ See Index 405.2.

④ Pedeistrain and bicycle features are not shown on figure.

405.7 Public Road Intersections

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

(1) *Right-turn Onto the Main Highway*. The combination of a circular curve joined by a 2:1 taper on the crossroads and a 75-foot taper on the main highway is designed to fit the wheel paths of the appropriate turning template chosen by the designer.

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

(2) *Right-turn Off the Main Highway*. The combination of a circular curve joined by a 150-foot taper on the main highway and a 4:1 taper on the crossroads is designed to fit the wheel paths of the appropriate turning template and to move the rear of the vehicle off the main highway. Deceleration and storage lanes may be provided when necessary (see Index 405.3).

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

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

    Intersections are major consideration in bicycle path design as well. See Indexes 403.6 and 1003.1(4) for general bicycle path intersection design guidance. Also see Section 5.3 of the AASHTO Guide for the Planning, Design, and Operation of Bicycle Facilities.

405.8 City Street Returns and Corner Radii

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

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

Smaller radii of 15 feet to 25 feet are appropriate at minor cross streets where few trucks or buses are turning. Local agency standards may be appropriate in urban and suburban areas.

Encroachment into opposing traffic lanes must be avoided.

405.9 Widening of 2-lane Roads at Signalized Intersections

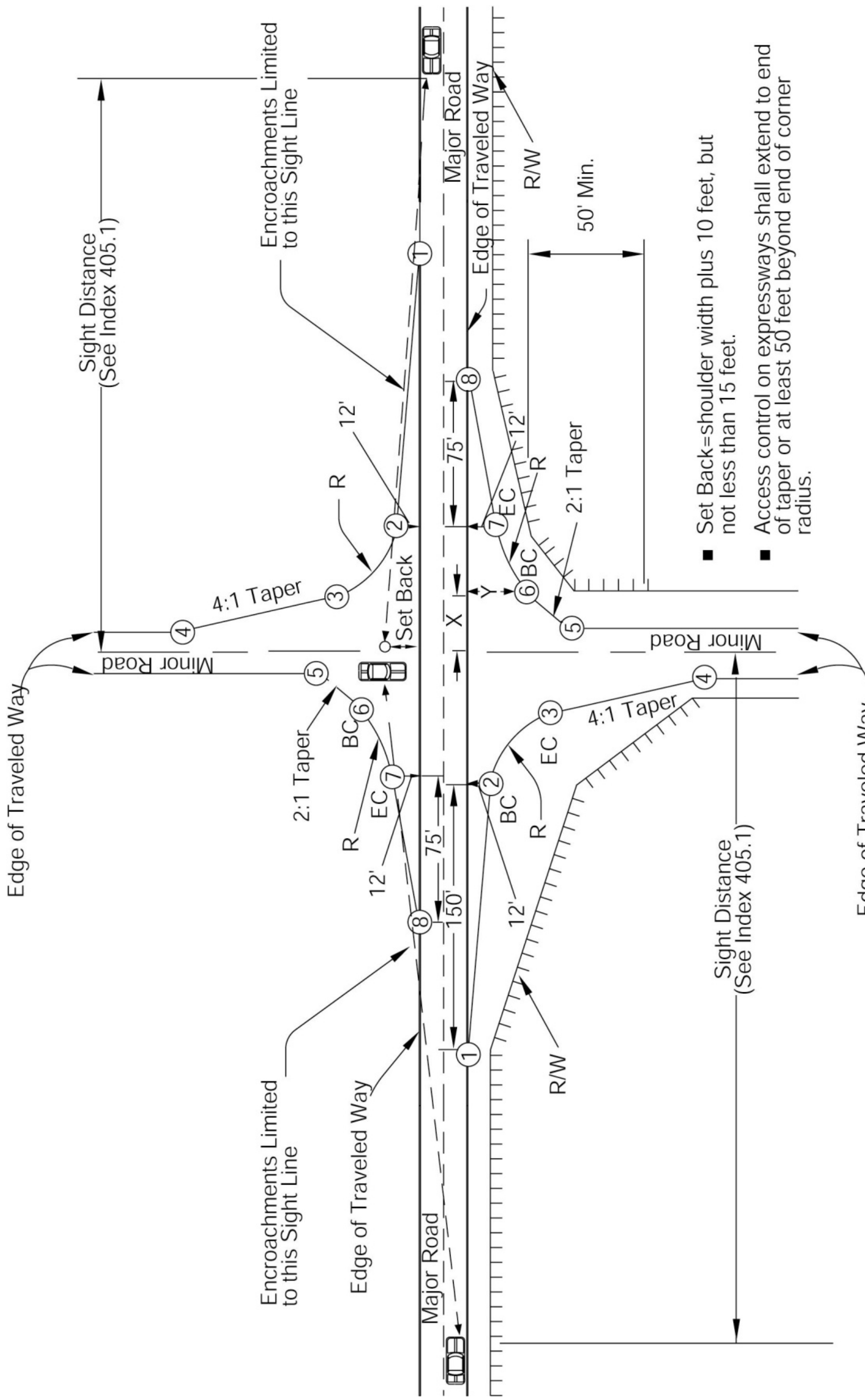
Two-lane State highways may be widened at intersections to 4-lanes whenever signals are installed. Sometimes it may be necessary to widen the intersecting road. The minimum design is shown in Figure 405.9. More elaborate treatment may be warranted by the volume and pattern of traffic movements. Unusual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

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

405.10 Roundabouts

Roundabout intersections on the State highway system must be developed and evaluated in accordance with National Cooperative Highway Research Program (NCHRP) Report 672 entitled “Roundabouts: An Informational Guide, 2nd ed.” (NCHRP Guide 2) dated October 2010 and Traffic Operations Policy Directive (TOPD) Number 13-02. Also see Index 401.5 for general information and guidance. See Figure 405.10 Roundabout Geometric Elements for nomenclature associated with roundabouts. Signs, striping and markings at roundabouts are to comply with the California MUTCD.

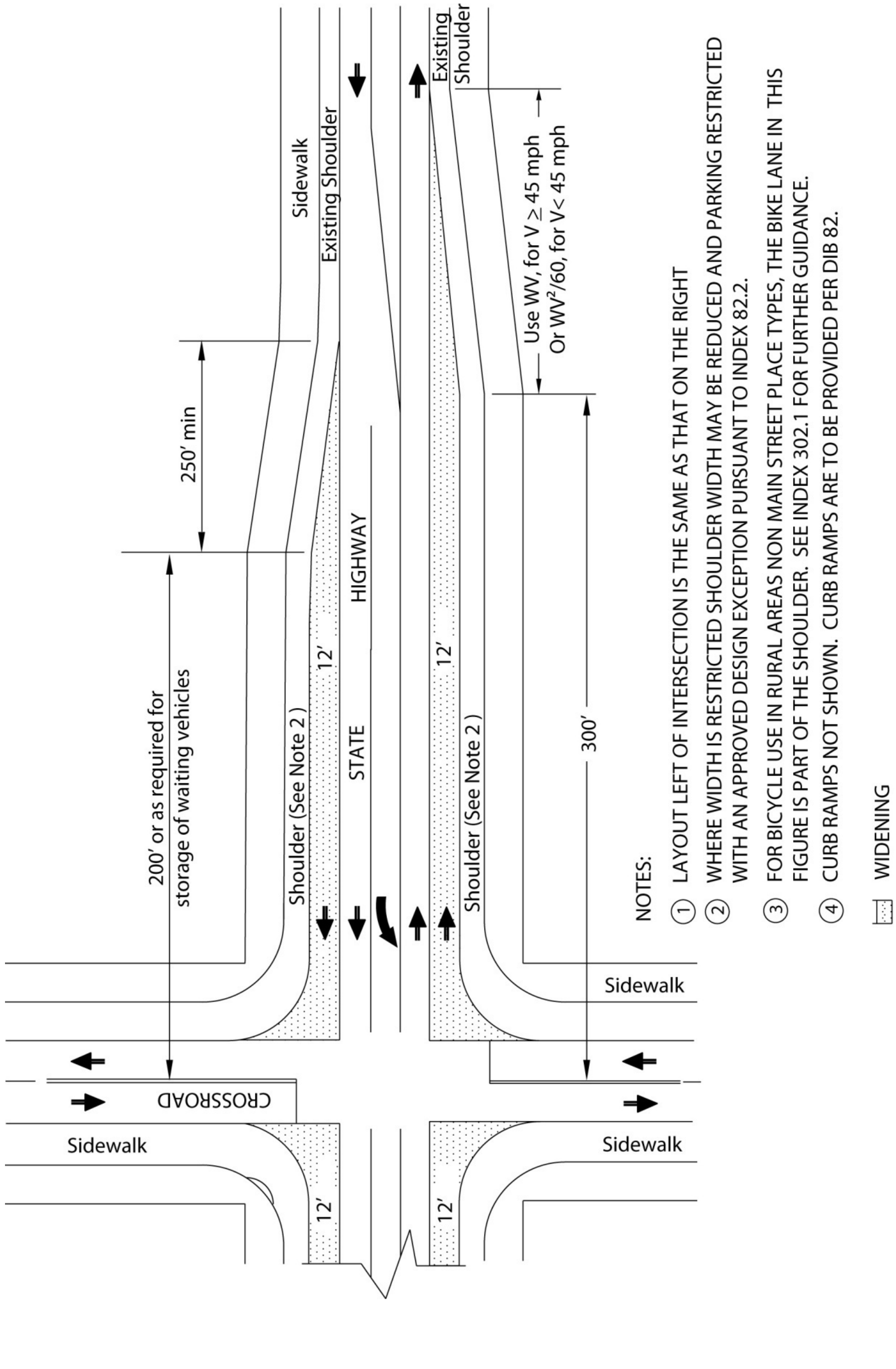
Figure 405.7
Public Road Intersections



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

Radius of Curve	Design Vehicle	Pt ①		Pt ②		Pt ③		Pt ④		Pt ⑤		Pt ⑥		Pt ⑦		Pt ⑧	
		X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
30'	Bus	204.20	0.0	54.20	12.0	27.49	34.63	12.0	96.58	12.0	40.66	18.23	28.21	40.32	12.0	115.32	0.0
40'	California	215.08	0.0	65.08	12.0	29.46	42.17	12.0	112.03	12.0	53.35	21.87	33.61	51.33	12.0	126.33	0.0
50'	STAA	226.09	0.0	76.09	12.0	31.57	49.71	12.0	127.98	12.0	75.63	30.31	39.01	67.13	12.0	142.13	0.0

Figure 405.9
Widening of Two-lane Roads at Signalized Intersections



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

(a) Speed reduction at the entry and through the intersection will be achieved through geometric design and,

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

Benefits of roundabouts are:

- Fewer conflict points typically result in fewer collisions with less severity. Over half of vehicle to vehicle points of conflict associated with intersections are eliminated with the use of a roundabout. Additionally, a roundabout separates the points of conflict which eases the ability of the users to identify a conflict and helps prevent conflicts from becoming collisions.
- Roundabouts are designed to reduce the vehicular speeds at intersections. Lower speeds lessens the vehicular collision severity. Likewise, studies indicate that pedestrian and bicyclist collisions with motorized vehicles at lower speeds significantly reduce their severity.
- Roundabouts allow continuous free flow of vehicles and bicycles when no conflicts exist. This results in less noise and air pollution and reduces overall delays at roundabout intersections.

Except as indicated in this Index, the standards elsewhere in this manual do not apply to roundabouts. For the application of design standards, the approach ends of the splitter islands define the boundary of a roundabout intersection, see Figure 405.10. The design standards elsewhere in this manual apply to the approach legs beyond the approach ends of the splitter islands.

(1) *Design Period.*

The design period guidance provided in Index 103.2 applies to roundabouts. When staging improvements, see NCHRP Guide 2, Section 6.12.

(2) *Design Vehicles* - See Topic 404.

The turning path for the design vehicle, see Index 404.5, dictates many of the roundabout dimensions. The design vehicle tracking and swept width are to be used when designing all the entries and exits, where design vehicles are unrestricted (see Index 404.2), and the circulatory roadway. The percentage of trucks and their lane utilization is an important consideration on multilane roundabouts when determining if the design will allow trucks to stay within their own lane or encroach into the adjacent lane. If permit vehicles larger than the design vehicle occasionally use the proposed roundabout, they can be accommodated by having removable signs or other removable features in the central island or around the circular path to ensure their swept path can negotiate the roundabout. Roundabouts should not be overdesigned for the occasional permit vehicle.

To accurately simulate the design vehicle swept width traveling through a roundabout, the minimum speed of the design vehicle used in computer simulation software (e.g., Auto TURN) should be 10 mph through the roundabout.

(3) *Inscribed Circle Diameter.*

At single lane roundabouts, the size of the inscribed circle is largely dependent upon the turning requirements of the design vehicle. The inscribed circle diameter must be large enough to accommodate: (a) the STAA design vehicle for all roundabouts on the National Network and on Terminal Access routes; and, (b) the California Legal design vehicle on all non-STAA route intersections on California Legal routes and California Legal KPRA Advisory routes, while maintaining adequate deflection curvature to ensure appropriate travel speeds for smaller vehicles. The design vehicle is to navigate the roundabout with the front tractor wheels off the truck apron, if one is present. Transit vehicles, fire engines and single-unit delivery vehicles are also to be able to navigate the roundabout without using the truck apron, if one is present. The inscribed circle diameter for a single lane roundabout generally ranges between 105 feet to 150 feet to accommodate the California Legal design vehicle and 130 feet to 180 feet to accommodate the STAA design vehicle.

At multilane roundabouts, the inscribed circle diameter is to achieve adequate alignment of the natural vehicle path while maintaining deflection curvature to ensure appropriate travel speeds. To achieve both of these design objectives requires a slightly larger diameter than used for a single lane roundabout. The inscribed circle diameter for a multilane (2-lane) roundabout generally ranges between 150 feet to 220 feet to accommodate the California Legal design vehicle for non-STAA route intersections on California Legal routes and California Legal KPRA Advisory routes, and 165 feet to 220 feet to accommodate the STAA design vehicle for roundabouts on the National Network and on Terminal Access routes. Similar to a single lane roundabout, the design vehicle is to be able to navigate a multilane roundabout with the front tractor wheels staying off the truck apron, if one is present. Transit vehicles, fire engines and single-unit delivery vehicles are also to be able to navigate the roundabout without using the truck apron, if one is present.

(4) *Entry Speeds.*

Lowering the speed of vehicles entering and traveling through the roundabout is a primary design objective that is achieved by approach alignment and entry geometry.

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

- <u>Single lane roundabouts, 25 mph.</u>
- <u>Multilane roundabouts, 30 mph.</u>

For fastest path evaluation, see NCHRP Guide 2, Section 6.7.1.

(5) *Exit Design.*

Similar to entry design, exit design flexibility is required to achieve the optimal balance between competing design variables and project objectives to provide adequate capacity and, essentially, safety while minimizing excessive property impacts and costs. Thus, the selection of a curved versus tangential design is to be based upon the balance of each of these criteria. Exit design is influenced by the place type, pedestrian demand, bicyclist needs, the design vehicle and physical constraints. The exit curb radii are usually larger than the entry curb radii in order to minimize the likelihood of congestion and crashes at the exits. However, the desire to minimize congestion at the exits needs to be balanced with the need to maintain an appropriate operating speed through the pedestrian crossing. Therefore, the exit path radius should not be significantly greater than the circulating path radius to ensure low speeds are maintained at the pedestrian crossing.

(6) *Number of Legs Serving the Roundabout.*

Intersections with more than four legs are often difficult to manage operationally. Roundabouts are a proven traffic control device in such situations. However, it is necessary to ensure that the design vehicle can maneuver through all unrestricted legs of the roundabout.

(7) *Pedestrian Use.*

Sidewalks around the circular roadway are to be designed as shared-use paths, see Index 405.10(8)(c). However, the guidance in Design Information Bulletin (DIB) 82 Pedestrian Accessibility Guidelines for Highway Projects must also be followed when designing these shared-use facilities around a roundabout. If there is a difference in the standards, the guidance in DIB 82 is to be followed. In addition,

(a) Pedestrian curb ramps need to be differentiated from bike ramps:

- The grooved border differentiates a pedestrian curb ramp from a bicycle ramp. Bicycle ramps for the use of bicyclists are not to utilize a grooved border.
- Detectable warning surface (truncated domes) are required on curb ramps.

They are not to be used on a bike ramp.

(b) Truck aprons and mountable curbs are not to be placed in the pedestrian crossing areas.

(c) See the California MUTCD for the signs and markings used at roundabouts.

(8) Bicyclist Use.

(a) General. Bicyclists may choose to travel in the circular roadway of a roundabout by taking a lane, while others may decide to travel using the shared-use path to bypass the circular roadway. Therefore, the approach and circular roadways, as well as the shared-use path all need to be designed for the mobility needs of bicyclists. See the California MUTCD for the signs and markings used at roundabouts.

(b) Bicyclist Use of the Circular Roadway. Single lane roundabouts do not require bicyclists to change lanes in the circular roadway to select the appropriate lane for their direction of travel, so they tend to be comfortable for bicyclists to use. Even two-lane roundabouts, which may have straighter paths of travel that can lead to faster vehicular traveling speeds, appear to be comfortable for bicyclists that prefer to travel like vehicles. Roundabouts that have more than two circular lanes can create complexities in signing and striping (see the California MUTCD for guidance), and their operating speed may cause some bicyclists to decide to bypass the circular roadway and use the bicycle ramp that provides access to the shared-use path around the roundabout.

(c) Bicyclists Use of the Shared-Use Path. The shared-use path is to be designed using the guidance in Index 1003.1 for Class I Bikeways and in NCHRP Guide 2 Section 6.8.2.2. However, the accessibility guidance in DIB 82 must also be followed when designing these shared-use facilities around a roundabout. If there is a difference in the standards, the accessibility guidance in DIB 82 is to be followed to ensure the facility is accessible to pedestrians with disabilities.

Bicycle ramps are to be located to avoid confusion as curb ramps for pedestrians. Also see Index 405.10(7) for guidance on how to differentiate the two types of ramps. The design details and width of the ramp are also important to the bicyclist. Bicyclists approaching the bicycle ramp need to be provided the choice of merging left into the lane or moving right to use the bicycle ramp. Bicycle ramps should be placed at a 35 to 45 degree angle to the departure roadway and the sidewalk to enable the bicyclists to use the ramp and discourage bicyclists from entering the shared-use path at a speed that is detrimental to the pedestrians. The shared-use path should be designated as Class I Bikeways; however, appropriate regulatory signs may need to be posted if the local jurisdiction has a law(s) that prohibit bicyclists from riding on a sidewalk.

A landscape buffer or strip between the shared-use/Class I Bikeway and the circular roadway of the roundabout is needed and should be a minimum of 2 feet wide.

Pedestrian crossings may also be used by bicyclists; thus, these shared-use crossings need to be designed for both bicyclist and pedestrian needs.

(9) Transit Use.

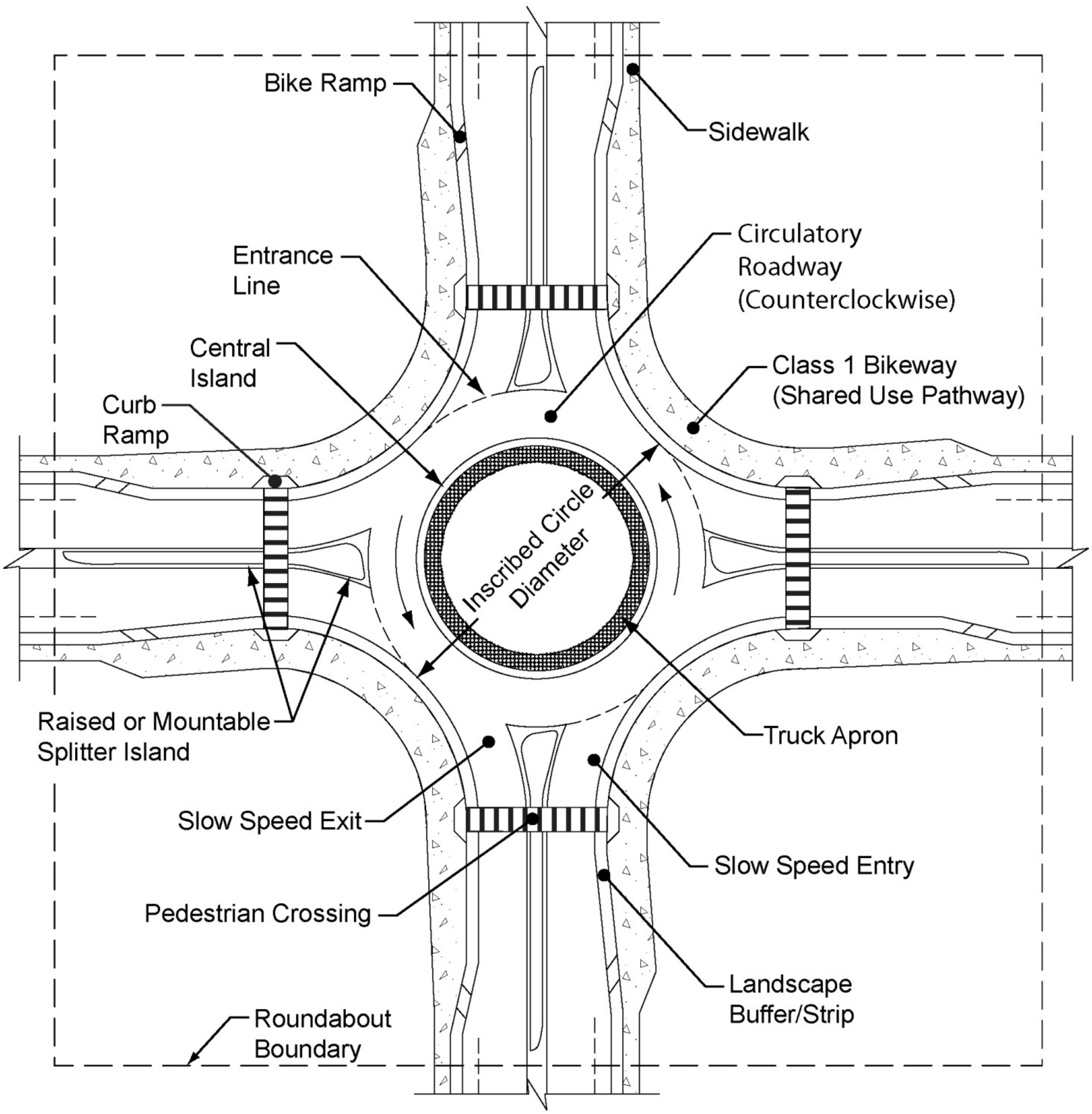
Transit vehicles and buses will not have difficulty negotiating a roundabout when it has been designed using the California Legal design vehicle or the STAA design vehicle. However, to minimize passenger discomfort, a roundabout should be designed such that the transit vehicle or bus does not use the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

(10) Stopping Sight Distance and Visibility.

See Index 201.1 for stopping sight distance guidance at roundabouts.

It is desirable to create a domed or mounded central island, between 3.5 to 6 feet high, to

Figure 405.10
Roundabout Geometric Elements



NOTE:

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

focus attention on the approach and through roundabout alignment. A domed central island provides a visual screen from downstream alignment and other distractions.

(11) *Speed Consistency.*

Consistency in operating speeds between the various movements within the roundabout can minimize collisions between traffic streams. The operating speeds between competing traffic streams and between consecutive geometric elements should be minimized such that the maximum speed differential between them is no more than 15 mph; it is preferred that the operating speed differential be less than 10 mph.

(12) *Path Alignment (Natural Path).*

As two traffic streams approach the roundabout in adjacent lanes, drivers and bicyclists will be guided by lane markings up to the entrance line. At the yield point, they will continue along their natural trajectory into the circulatory roadway. The speed and orientation of the design vehicle at the entrance line determines what can be described as its natural path. The geometry of the exits also affects the natural path that the design vehicle travels. The natural path of two vehicles are not to overlap, see NCHRP Guide 2, Section 6.7.2.

(13) *Splitter Islands.*

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

The total length of the raised island should be at least 50 feet although 100 feet is desirable. On higher speed roadways, splitter island lengths of 150 feet or more is beneficial. Additionally, the splitter island should extend beyond the end of the exit curve to prevent exiting traffic from crossing into the path of approaching traffic. The splitter island width should be a minimum of 6 feet at the pedestrian crossing to adequately provide refuge for pedestrians.

Posted speeds on the approach roadway greater than or equal to 45 mph require the splitter island length, as measured from the inscribed circle diameter, to be 200 feet. In some instances, a longer splitter island may be desirable. Concrete curb is to be provided on the right side of the approach roadway equal to the length of the splitter island from the inscribed circle diameter.

(14) *Access Control.*

The access control standards in Index 504.3(3) and 504.8 apply to roundabouts at interchange ramp intersections. The dimensions shown in Index 504.8 are to be measured from the inscribed circle diameter.

Driveways should not be placed within 100 feet from the inscribed circle diameter.

(15) *Lighting.*

Lighting is required at all roundabouts. See the Traffic Manual Chapter 9 as well as consult with the District Traffic Operations Branch.

(16) *Landscaping.*

Landscaping should be designed such that drivers and bicyclists can observe the signing and shape of the roundabout as they approach, allowing adequate visibility for making decisions within the roundabout. The landscaping of the central island can enhance the intersection by making it a focal point, by promoting lower speeds and by breaking the headlight glare of oncoming vehicles or bicycles. It is desirable to create a domed or mounded central island, between 3.5 to 6 feet high, to increase the visibility of the intersection on the approach. Contact the District Landscape Architecture Unit to provide technical assistance in designing the roundabout landscaping.

(17) *Vertical Clearance.*

The vertical clearance guidance provided in Index 309.2 applies to roundabouts.

(18) Drainage Design.

See Chapter 800 to 890 for further guidance.

Topic 406 - Ramp Intersection Capacity Analysis

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

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

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

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

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

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

Table 406

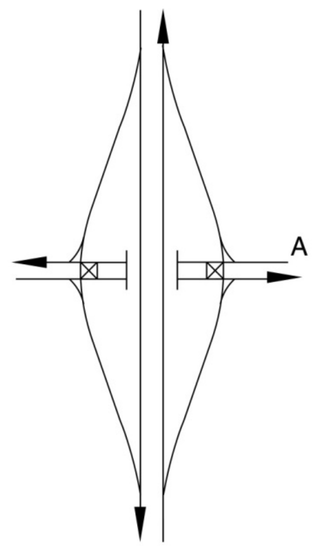
Vehicle Traffic Flow Conditions at Intersections at Various Levels of Operation

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

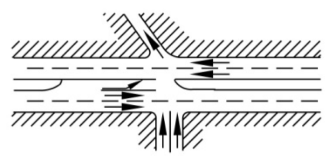
NOTE:

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

Figure 406A
Spread Diamond



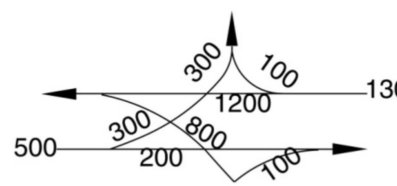
A "spread" diamond, where storage is available between ramp intersections.



LOCATION A

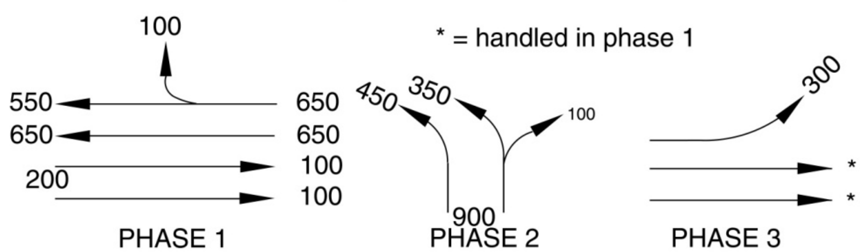
TRAFFIC FLOWS

NOTE: Traffic from field counts, A.M. peak



LANE VOLUMES (ILV/Hr)

* = handled in phase 1



Identify and Sum-up Critical Lane Volumes

PHASE 1: 650 ~~100~~ → 650 ILV/Hr.

PHASE 2: 450 → 450 ILV/Hr.

PHASE 3: ~~*~~ 300 → 300 ILV/Hr.

Total - - - - - 1400 ILV/Hr.

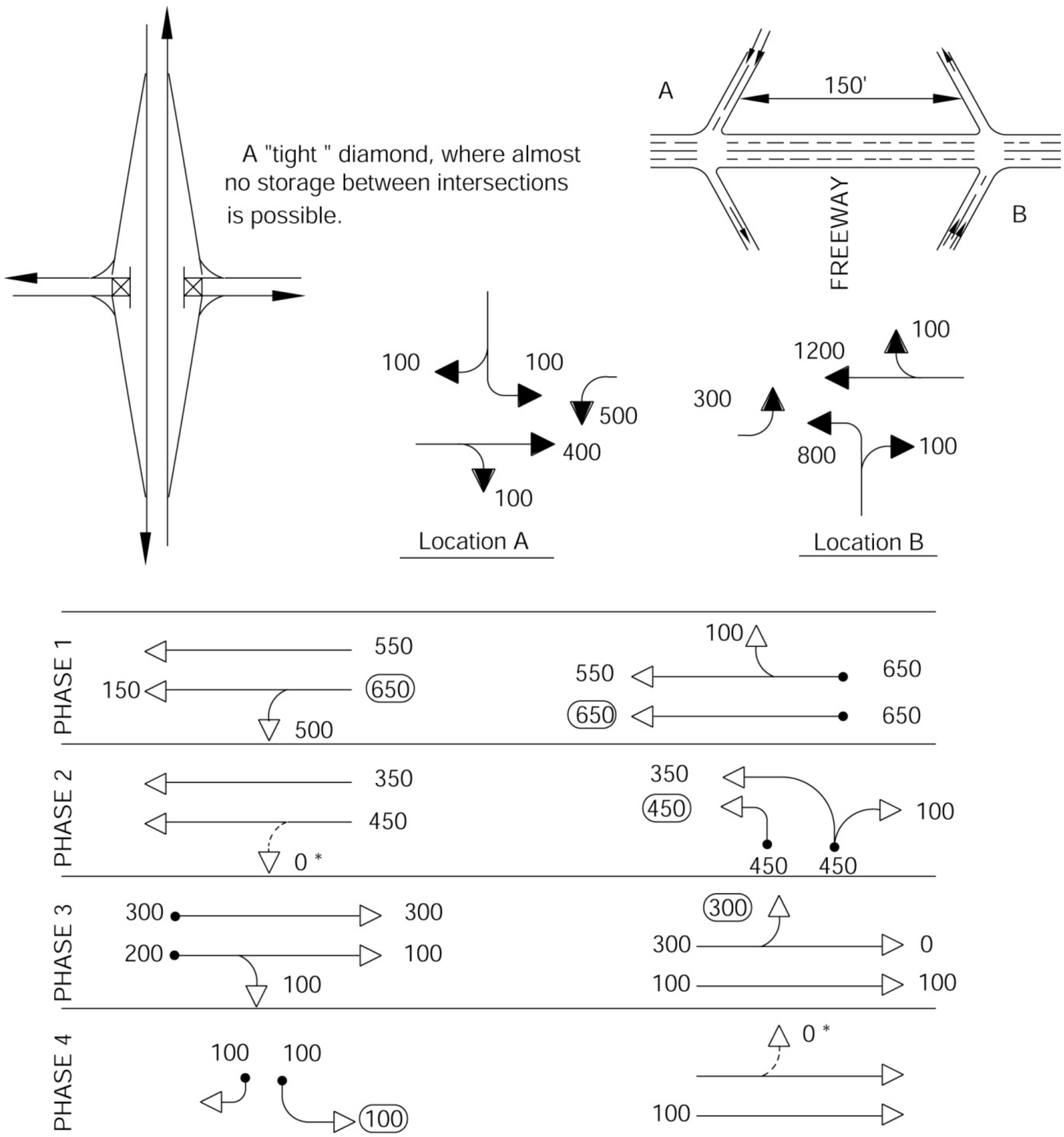
Evaluate Operating Level: (1400 ILV/Hr.)

1200 < 1400 < 1500

The total volume of traffic which shares the intersection does not exceed capacity (1500), but is greater than 1200 ILV/Hr. threshold.
This suggests that congestion would be present and the intersection would be approaching capacity.

ILV = Intersecting Lane Vehicles.

Figure 406B
Tight Diamond



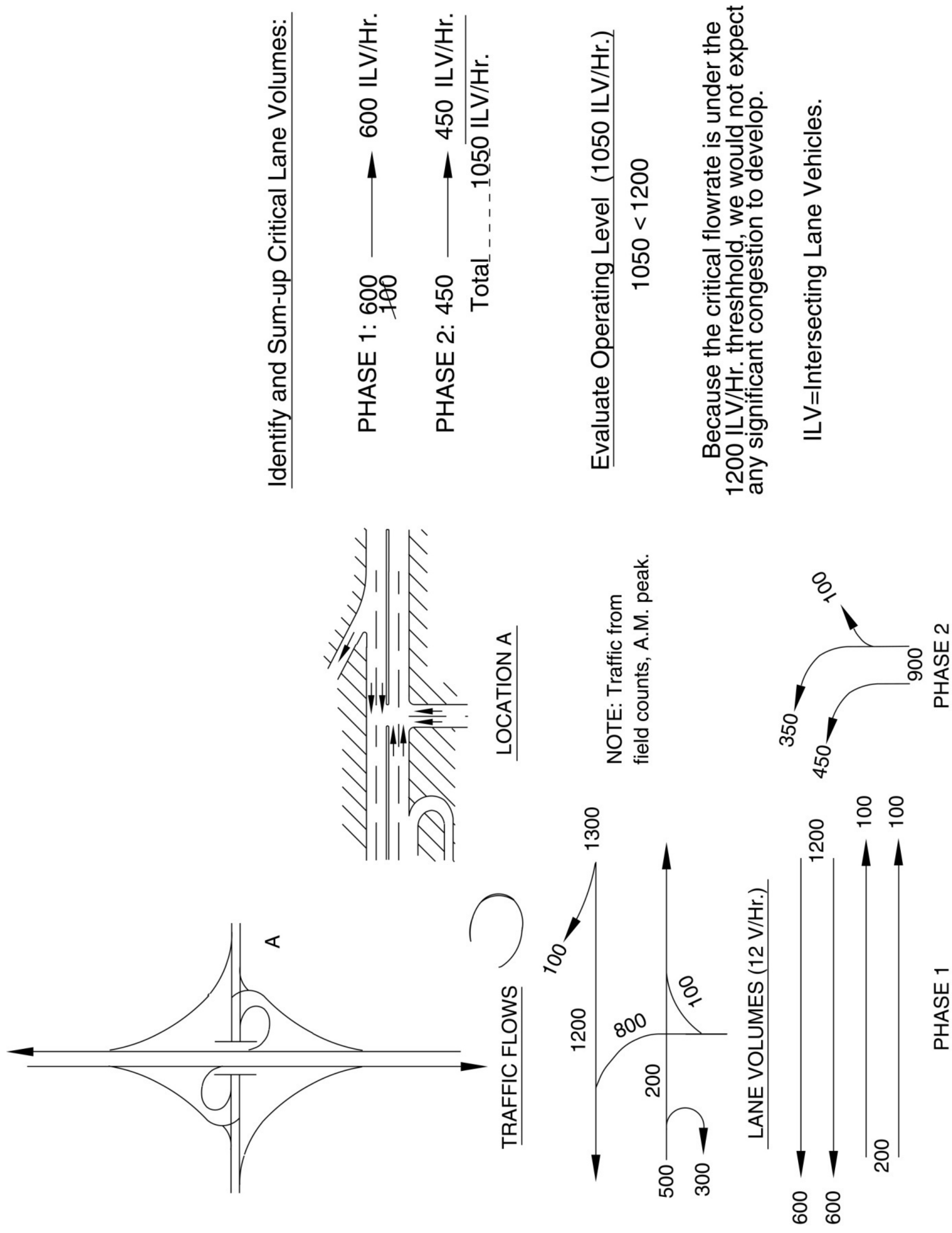
*NOTE: When no storage at all is permitted, left-turn movement is cleared during this phase.

Critical Lane Volumes:

650
450
300
100
1500 ILV/Hr.

ILV=Intersecting Lane Vehicles.

Figure 406C
Two-quadrant Cloverleaf



elevated and the cross street retains a straight profile. Type L-1's are suitable where physical, geometric or right of way restrictions do not permit a spread diamond configuration. Compact diamonds have the disadvantage of requiring wider overcrossing or longer span undercrossing to provide corner sight distance and have limited capacity between intersections. Once the area around the interchange is developed, Type L-1 is challenging to expand to accommodate growth.

The spread diamond (Type L-2) is adaptable where the grade of the cross street is changed to pass over or under the freeway. The ramp terminals are spread in order to achieve maximum sight distance and minimum intersection cross slope, commensurate with construction and right of way costs, travel distance, and general appearance. A spread diamond has the advantage of flatter ramp grades, greater crossroads left-turn storage capacity, and the flexibility of permitting the construction of future loop ramps if required.

The split diamond with braids (Type L-3) may be appropriate where two major crossroads are closely spaced.

(b) Interchanges with Parallel Street Systems--Types L-4, L-5 and L-6 are interchange systems used where the freeway alignment is placed between parallel streets. Types L-4 and L-5 are used where the parallel streets will operate with one-way traffic. In Type L-4 slip ramps merge with the frontage street and in Type L-5 the ramps terminate at the intersection of the frontage road with the cross street, forming five-legged intersections. In Type L-6 the freeway ramps connect with two-way parallel streets. The parallel streets in the Types L-4, L-5 and L-6 situation are usually too close to the freeway to permit ramp intersections on the cross street between the parallel frontage streets.

The "hook" ramps of the Type L-6 are often forced into tight situations that lead to less than desirable geometrics. The radius of the curve at the approach to the intersection should exceed 150 feet and a tangent of at least 150 feet should be provided between the last curve on the ramp and the ramp terminal.

Special attention should always be given to exit ramps that end in a hook to ensure that adequate sight distance around the curve, adequate deceleration length prior to the curve or end of anticipated queue, and adequate superelevation for anticipated driving speeds can be developed. Type L-6 can only be considered when all other interchange types are not acceptable.

(c) Cloverleaf Interchanges--The simplest cloverleaf interchange is the two-quadrant cloverleaf, Type L-7 or Type L-8, or a combination where the two loops are on the same side of the cross street. Type L-7 eliminates the need for left-turn storage lanes, on or under the structure, thus reducing the structure costs. These interchanges should be used only in connection with controls which preclude the use of diamond ramps in all four quadrants. These controls include right of way controls, a railroad track paralleling the cross street, and a short weaving distance to the next interchange.

The Type L-9, partial cloverleaf interchange, provides loop on-ramps in addition to the four diamond-type ramps. This interchange is suitable for large volume turning movements. Left-turn movements from the crossroads are eliminated, thereby permitting two-phase operation at the ramp intersections when signalized. Because of this feature, the Type L-9 interchange usually has capacity to handle the higher volume traffic on the crossroad.

The four-quadrant cloverleaf interchange (Type L-10) offers free-flow characteristics for all movements. It has the disadvantage of a higher cost than a diamond or partial cloverleaf design, as well as a relatively short weaving section between the loop ramps which limits capacity. For this reason this type of interchange is not desirable. Collector-distributor roads should be incorporated in the design of four-quadrant cloverleaf interchanges to separate the weaving conflicts from the through freeway traffic.

(d) Trumpet Interchanges--A trumpet design, Type L-11 or L-12, may be used when a crossroads terminates at a freeway. This design should not be used if future extension of the crossroads is probable. The diamond interchange is preferable if future extension of the crossroads is expected.

(e) Single Point Interchange (SPI)--The Type L-13 is a concept which essentially combines two separate diamond ramp intersections into one large at-grade intersection. It is also known as an urban interchange. Additional information on SPI's is provided in the Single Point Interchange Planning, Design and Operational Guidelines (SPI Guidelines), issued by memorandum on June 15, 2001.

Type L-13 requires approximately the same right of way as the compact diamond. However, the construction cost is substantially higher due to the structure requirements. The capacity of the L-13 can exceed that of a compact diamond if long signal times can be provided and left turning volumes are balanced.

This additional capacity may be offset if nearby intersection queues interfere with weaving and storage between intersections. The disadvantages of the L-13 are: 1) future expansion of the interchange is extremely difficult; 2) stage construction for retrofit situations is costly; 3) long structure spans require higher than normal profiles and deeper structure depths; and 4) poor bicycle and pedestrian circulation.

(f) Other Types of Interchanges--New or experimental interchanges must have the Design Coordinator and Traffic Liaison's concurrence before selection. Concurrence may require additional studies and documentation.

502.3 Freeway-to-Freeway Interchanges

(1) General. The function of the freeway-to-freeway interchange is to link freeway segments together so as to provide the highest level of service in terms of mobility. Parameters such as cost, environment, community values, traffic volumes, route continuity, driver expectation and safety should all be considered. Route continuity, providing for the designated route to continue as the through movement through an interchange, reduces lane changes, simplifies signing, and reduces driver confusion. **Interstate routes shall maintain route continuity. Where both the designated route and heavier traffic volume route are present, the interchange configuration shall keep the designated route to the left through the interchange.**

(2) Design Considerations.

(a) Cost--The differential cost between interchange types is often significant. A cost-effective approach will tend to assure that an interchange is neither over nor under designed. Decisions as to the relative values of the previously mentioned parameters must be consistent with decisions reached on adjacent main line freeways.

(b) System Balance--The freeway-to-freeway interchange is a critical link in the total freeway system. The level of traffic service provided will have impact upon the mobility and overall effectiveness of the entire roadway system. For instance, traffic patterns will adjust to avoid repetitive bottlenecks, and to the greatest degree possible, to temporary closures, accidents, etc. The freeway-to-freeway interchange should provide flexibility to respond to these needs so as to maximize the cost effectiveness of the total system.

(c) Provide for all Traffic Movements--All interchanges must provide for each of the eight basic movements (or four basic movements in the case of a three-legged interchange), except in the most extreme circumstances. Less than "full interchanges" may be considered on a case-by-case basis for applications requiring special access for managed lanes (e.g., transit, HOVs, HOT lanes) or park and ride lots. Partial interchanges usually have undesirable operational characteristics. If circumstances exist where a partial interchange is considered appropriate as an initial phase improvement, then commitments need to be included in the request to accommodate the ultimate design. These commitments may include purchasing the right of way required during the initial phase improvements.

(d) Local Traffic Service--In metropolitan areas a freeway-to-freeway interchange is usually superimposed over an existing street system. Local and through traffic requirements are often in conflict.

Combinations of local and freeway-to-freeway interchanges can result in designs that are both costly and so complex that the important design concepts of simplicity and consistency are compromised. Therefore, alternate plans separating local and freeway-to-freeway interchanges should be fully explored. Less than desirable local interchange spacing may result; however, this may be compensated for by upgrading the adjacent local interchanges and street system.

Local traffic service interchanges should not be located within freeway-to-freeway interchanges unless geometric standards and level of service will be substantially maintained.

(e) Alignment--It is not considered practical to establish fixed freeway-to-freeway interchange alignment standards. An interchange must be designed to fit into its environment. Alignment is often controlled by external factors such as terrain, buildings, street patterns, route adoptions, and community value considerations. Normally, loops have radii in the range of 150 feet to 200 feet and direct connections should have minimum radii of 850 feet. Larger radii may be proper in situations where the skew or other site conditions will result in minimal increased costs. Direct connection radii of at least 1,150 feet are desirable from a traffic operational standpoint. High alignment and sight distance standards should be provided where possible.

Drivers have been conditioned to expect a certain standard of excellence on California freeways. The designer's challenge is to provide the highest possible standards consistent with cost and level of service.

(3) Types. Several freeway-to-freeway interchange design configurations are shown on Figure 502.3. Many combinations and variations may be formed from these basic interchange types.

(a) Four-Level-Interchange--Direct connections are appropriate in lieu of loops when required by traffic demands or other specific site conditions. The Type F-1 interchange with all direct connections provides the maximum in mobility and safety. However, the high costs associated with this design require that the benefits be fully substantiated.

The Type F-1 Alternative "A" interchange utilizes a single divergence ramp for traffic bound for the other freeway; then provides a secondary directional split. Each entrance ramp on a Type F-1A interchange is provided separately. The advantages of the Type F-1A are: 1) reduced driver confusion since there is only one exit to the other freeway, and 2) operations at the entrance may be improved since the ramps merge with the mainline one at a time.

The Type F-1 Alternative "B" interchange provides separate directional exit ramps and then merges the entering traffic into a single ramp before converging with the mainline. Since the Type F-1B combines traffic from two ramps before entering the freeway, it is important to verify that adequate weaving capacity is provided beyond the entrance. Separating the directional split of exiting traffic reduces the volume to each of the two ramps and therefore may improve the level of service of the weave section prior to the exit.

Design for a four-level interchange may combine the configuration of the Type F1-A and F1-B interchange to best suit the conditions at a given location.

(b) Combination Interchanges--The three-quadrant cloverleaf, Type F-2, with one direct connection may be necessary where a single move carries too much traffic for a loop ramp or where the one quadrant is restricted by environmental, topographic, or right of way controls.

The two-loop, two-direct connection interchange, Type F-3, is often an appropriate solution. The weaving conflicts which ordinarily constitute the most restrictive traffic constraint are eliminated, yet cost and right of way requirements may be kept within reasonable bounds. Consideration should be given to providing an auxiliary lane in advance of the loop off-ramps to provide for vehicle deceleration.

(c) Four-Quadrant Cloverleaf--The four-quadrant cloverleaf with collector-distributor roads, Type F-4, is ordinarily the most economical freeway-to-freeway interchange solution when all turning movements are provided. The four-quadrant cloverleaf is generally applicable in situations where turning volumes are low enough to be accommodated in the short weaving sections. It should be designed with collector-distributor roads to separate weaving conflicts from the through freeway traffic.

(d) Freeway Terminal Junction--Types F-5, F-6, F-7, and F-8 are examples of interchange designs where one freeway terminates at the junction with another freeway. In general, the standard of alignment provided on the left or median lane connection from the terminating freeway should equal or approach as near as possible that of the terminating freeway. Terminating the median lane on a loop should be avoided. It is preferable that both the designated route and the major traffic volume be to the left at the branch connection diverge. The choice between Types F-7 and F-8 should include considerations of traffic volumes, and route continuity. When these considerations are in conflict, the choice is made on the basis of judgment of their relative merits.

Topic 503 - Interchange Design Procedure

503.1 Basic Data

Data relative to community service, traffic, physical and economic factors, and potential area development which may materially affect design, should be obtained prior to interchange design. Specifically, the following information should be available:

(a) The location and standards of existing and proposed local streets including types of traffic control.

(b) Existing, proposed and potential for development of land, including such developments as employment centers, retail services and shopping centers, recreational facilities, housing developments, schools, and other institutions.

(c) A vehicle traffic flow diagram showing average daily traffic and design hourly volumes, as well as time of day (a.m. or p.m.), anticipated on the freeway ramps and affected local streets or roads.

(d) Current and future bicycle and pedestrian access through the community.

(e) The relationship with adjacent interchanges.

(f) The location of major utilities, railroads, or airports.

(g) The presence of dedicated lanes and associated ramps and connections, including HOV lanes, Bus (BRT) lanes and Express lanes.

(h) The planned ultimate build-out for the freeway facility.

(i) Existing and planned rail facilities.

503.2 Reviews

Interchanges are among the major design features which are to be reviewed by the Design Coordinator and/or Design Reviewer, HQ Traffic Liaison, other Headquarters staff, and the FHWA Transportation Engineer, as appropriate. Major design features include the freeway alignment, geometric cross

(j) Inlet and outlet treatment.

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

801.6 Use of Drainage References

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

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

Topic 802 - Drainage Design Responsibilities

802.1 Functional Organization

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

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

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

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

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

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

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

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

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

(b) The structural adequacy of all drainage facilities.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

- Storm drain design and calculations.
- Drainage basins exceeding 320 acres.
- Hydrograph development or routing.
- Open channel modification or realignment.
- Retention or detention basins.
- Backwater analysis.
- High potential for flood damage litigation.
- Scour analysis or sediment transport (typically forwarded to DOS).
- Culvert designs greater than 36 inches in diameter.
- Encroachments on FEMA designated floodplains.
- Modifications to inlet or outlet capacities on existing culverts or drainage inlets (e.g., placement of safety end grates, conversion of side opening inlets to grated inlets, etc.).

normally the responsibility of the Environmental Planning Branch. The District Hydraulics Engineer will, as necessary, develop the hydrological and hydraulic information and provide technical assistance for assessing impacts of floodplain encroachments.

804.7 Preliminary Evaluation of Risks and Impacts for Environmental Document Phase

Virtually all proposed highway improvements that are considered as floodplain encroachments will be designed to have:

(a) No significant risks associated with implementation and,

(b) Negligible environmental impacts on the base floodplain.

(1) Risks. There will always be some potential for property damage and flooding that may affect public safety, associated with highway drainage design. In a majority of cases, a field review with a NFIP or USGS map and the application of good engineering judgment are all that is needed to determine if such risks are significant or acceptable. The detail of study and documentation shall be commensurate with the risk(s) or floodplain impact(s) and, in all cases, should be held to the minimum necessary to address 23 CFR 650.111.

(2) Impacts. The assessment of potential impacts on the floodplain environment will include:

(a) Impacts on natural and beneficial floodplain values.

(b) Support of probable incompatible floodplain development.

Except for the more environmentally sensitive projects, a single visit to the project site by the District Project Engineer, Hydraulics Engineer, and Environmental Planner, to assess and document the risks and environmental impacts associated with the proposed project is generally all that is necessary to obtain enough information for the "Location Hydraulic Study". Any reasonable adaptation of the technical information for “Location Hydraulic Study” form, Figure 804.7A, may be utilized to document and summarize the findings of the "Location Hydraulic Study" when the project is expected to be processed with a categorical exclusion. Items listed in 23 CFR 650.111 as follows must be addressed:

(a) National Flood Insurance Program (NFIP) maps or information developed by the highway agency, if NFIP maps are not available, shall be used to determine whether a highway location alternative will include an encroachment.

(b) Location studies shall include evaluation and discussion of the practicability of alternatives to any longitudinal encroachments.

(c) Location studies shall include discussion of the following items, commensurate with the significance of the risk or environmental impact, for all alternatives containing encroachments and for those actions which would support base floodplain development:

(1) The risks associated with implementation of the action,

(2) The impacts on natural and beneficial floodplain values,

(3) The support of probable incompatible floodplain development,

(4) The measures to minimize floodplain impacts associated with the action, and

(5) The measures to restore and preserve the natural and beneficial floodplain values impacted by the action.

(d) Location studies shall include evaluation and discussion of the practicability of alternatives to any significant encroachments or any support of incompatible floodplain development.

(e) The studies required by Sec. 650.111 (c) and (d) shall be summarized in environmental review documents prepared pursuant to 23 CFR part 771.

(f) Local, State, and Federal water resources and floodplain management agencies should be consulted to determine if the proposed highway action is consistent with existing watershed and floodplain management programs and to obtain current information on development and proposed actions in the affected watersheds.

Figure 804.7A is considered the suggested minimum hydraulic and engineering documentation for floodplain encroachments (bridge, culvert, channel change, slope protection, embankment, etc.). It is intended as a guide tool to help address the items listed in 23 CFR 650.111 and should be prepared jointly by the Project Engineer and Hydraulics Engineer. Since every location is unique, some of the questions may not apply, or additional considerations may need to be added.

For projects requiring an Environmental Impact Statement or Environmental Assessment (EIS/EA) or a finding of no significant impact (FONSI) with alternatives that have permanent features that encroach on the floodplain, a back-up report entitled Floodplain Evaluation is normally prepared by the District Environmental Branch. The technical requirements are typically developed jointly by the District Project Engineer and District Hydraulics Engineer. See Figure 804.7B for the Floodplain Evaluation Report Summary form that is used when an environmental document is to be prepared.

804.8 Design Standards

The design standards for highways encroaching on a floodplain are itemized in 23 CFR, Section 650.115. One requirement often overlooked is the need to assess the costs and risks associated with the overtopping flood for design alternatives in those instances where the overtopping flood exceeds the base flood. The content of design study information to be retained in the project file are described in 23 CFR, Section 650.117.

804.9 Coordination with the Local Community

The responsibility for enforcing National Flood Insurance Program (NFIP) regulations rests with the local community that is participating in the NFIP. It is the community who must submit proposals to Federal Emergency Management Agency (FEMA) for amendments to NFIP ordinances and maps in that community, or to demonstrate that an alternative floodway configuration meets NFIP requirements. However, this responsibility may be borne by the agency proposing to construct the highway crossing. Therefore, the highway agency should deal directly with the community and, through them, deal with FEMA. Determination of the status of a community's participation in the NFIP and review of applicable NFIP maps and study reports are, therefore, essential first steps in conducting location hydraulic studies and preparing environmental documents.

804.10 National Flood Insurance Program

The Flood Disaster Protection Act of 1973 (Pl 93-234, 87 Stat. 975) denies Federal financial assistance to flood prone communities that fail to qualify for flood insurance. The Act requires communities to adopt certain land use controls in order to qualify for flood insurance. These land use requirements could impose restrictions on the construction of highways in floodplains and regulatory floodplains in communities which have qualified for flood insurance.

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

In riverine situations, when the Administrator of the Federal Insurance Administration has identified the flood prone area, the community must require that, until a floodway has been designated, no use, including land fill, be permitted within the floodplain area having special flood hazards for which base flood elevations have been provided, unless it has been demonstrated that the cumulative effect of the proposed use, when combined with all other existing and reasonably anticipated uses of similar nature, will not increase the water surface elevation of the 100-year flood more than 1 foot at any point within the community.

Dike. (1) Usually an earthen bank alongside and parallel with a river or open channel or an AC dike along the edge of a shoulder. (See Levee) (2) An AC dike along the edge of a shoulder.

Dike, Finger. Relatively short embankments constructed normal to a larger embankment, such as an approach fill to a bridge. Their purpose is to impede flow and direct it away from the major embankment.

Dike, Toe. Embankment constructed to prevent lateral flow from scouring the corner of the downstream side of an abutment embankment. Sometimes referred to as training dikes.

Dike, Training. Embankments constructed to provide a transition from the natural stream channel or floodplain, both to and from a constricting bridge crossing.

Discharge. A volume of water flowing out of a drainage structure or facility. Measured in cubic feet per second.

Dissipate. Expend or scatter harmlessly, as of energy of moving water.

Ditch. Small artificial channel, usually unlined.

Diversion. (1) The change in character, location, direction, or quantity of flow of a natural drainage course (a deflection of flood water is not a diversion). (2) Draft of water from one channel to another. (3) Interception of runoff by works which discharge it thru unnatural channels.

D-Load (Cracking D-Load). A term used in expressing the strength of concrete pipe. The cracking D-load represents the test load required to produce a 0.01 inch crack for a length of 12 inches.

Downdrain. A prefabricated drainage facility assembled and installed in the field for the purpose of transporting water down steep slopes.

Downdrift. The direction of predominant movement of littoral materials.

Drain. Conduit intercepting and discharging surplus ground or surface water.

Drainage. (1) The process of removing surplus ground or surface water by artificial means. (2) The system by which the waters of an area are removed. (3) The area from which waters are drained; a drainage basin.

Drainage Area (Drainage Basin) (Basin). That portion of the earth's surface upon which falling precipitation flows to a given location. With respect to a highway, this location may be either a culvert, the farthest point of a channel, or an inlet to a roadway drainage system.

Drainage Course. Any path along which water flows when acted upon by gravitational forces.

Drainage Divide. The rim of a drainage basin. A series of high points from which water flows in two directions, to the basin and away from the basin.

Drainage Easement (See Easement).

Drainage System. Usually a system of underground conduits and collector structures which flow to a single point of discharge.

Drawdown. The difference in elevation between the water surface elevation at a constriction in a stream or conduit and the elevation that would exist if the constriction were absent. Drawdown also occurs at changes from mild to steep channel slopes and weirs or vertical spillways.

Drift. (1) Floating or non-mineral burden of a stream. (2) Deviation from a normal course in a cross current, as in littoral drift.

Drop. Controlled fall in a stream to dissipate energy.

Dry Weather Flows. A small amount of water which flows almost continually due to lawn watering, irrigation or springs.

Dune. A sand wave of approximately triangular cross section (in a vertical plane in the direction of flow) formed by moving water or wind, with gentle upstream slope and steep downstream slope and deposition on the downstream slope.

Easement. Right to use the land of others.

Ebb. Falling stage or outward flow, especially of tides.

Eddy. Rotational flow around a vertical axis.

Eddy Loss. The energy lost (converted into heat) by swirls, eddies, and impact, as distinguished from friction loss.

Embankment. Earth structure above natural ground.

Embayment. Indentation of bank or shore, particularly by progressive erosion.

Encroachment. Extending beyond the original, or customary limits, such as by occupancy of the river and/or floodplain by earth fill embankment.

Endwall. A wall placed at the end of a culvert. It may serve three purposes; (1), to hold the embankment away from the pipe and prevent sloughing into the pipe outlet channel; (2), to provide a wall which will prevent erosion of the roadway fill; and (3), to prevent flotation of the pipe.

Energy. Potential or kinetic, the latter being expressed in the same unit (feet) as the former.

Energy Dissipator. A structure for the purpose of slowing the flow of water and reducing the erosive forces present in any rapidly flowing body of water.

Energy Grade Line. The line which represents the total energy gradient along the channel. It is established by adding together the potential energy expressed as the water surface elevation referenced to a datum and the kinetic energy (usually expressed as velocity head) at points along the stream bed or channel floor.

Energy Head. The elevation of the hydraulic grade line at any section plus the velocity head of the mean velocity of the water in that section.

Entrance. The upstream approach transition to a constricted waterway.

Entrance Head. The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head.

Entrance Loss. The head lost in eddies and friction at the inlet to a conduit or structure.

Ephemeral. Of brief duration, as the flow of a stream in an arid region.

Equalizer. A drainage structure similar to a culvert but different in that it is not intended to pass a design flow in a given direction. Instead it is often placed level so as to permit passage of water in either direction. It is used where there is no place for the water to go. Its purpose is to maintain the same water surface elevation on both sides of the highway embankment.

Erosion. The wearing away of natural (earth) and unnatural (embankment, slope protection, structure, etc.) surfaces by the action of natural forces, particularly moving water and materials carried by it. In the case of drainage terminology, this term generally refers to the wearing away of the earth's surface by flowing water.

Erosion and *Scour*. The cutting or wearing away by the forces of water of the banks and bed of a channel in horizontal and vertical directions, respectively.

Erosion and *Accretion*. Loss and gain of land, respectively, by the gradual action of a stream in shifting its channel by cutting one bank while it builds on the opposite bank. Property is lost by erosion and gained by accretion but not by *avulsion* when the shift from one channel to another is sudden. Property is gained by *reliction* when a lake recedes.

Estuary. That portion of a river channel occupied at times or in part by both sea and river flow in appreciable quantities. The water usually has brackish characteristics.

Evaporation. A process whereby water as a liquid is changed into water vapor, typically through heat supplied from the sun.

Face. The outer layer of slope revetment.

Fan. A portion of a cone, but sometimes used to emphasize definition of radial channels. Also reference to spreading out of water or soils associated with waters leaving a confined channel (e.g., alluvial fan).

Fetch. The unobstructed distance across open water through which wind acts to generate waves.

Filter. A porous article or mass (as of fabric or even-graded mineral aggregate) through which water will freely pass, but which will block the passage of soil particles.

Filter Fabric (RSP fabric). An engineering fabric (geotextile) placed between the backfill and supporting or underlying soil through which water will pass and soil particles are retained.

Filter Layer. A layer of even-graded rock between rock riprap and underlying soil to prevent extrusion of the soil thru riprap.

Flap Gate. This is a form of valve that is designed so that a minimum force is required to push it open but when a greater water pressure is present on the outside of the valve, it remains shut so as to prevent water from flowing in the wrong direction. Construction is simple with a metal cover hanging from an overhead rod or pinion at the end of a culvert or drain.

Flood Frequency. Also referred to as exceedance interval, recurrence interval or return period; the average time interval between actual occurrences of a hydrological event of a given or greater magnitude; the percent chance of occurrence is the reciprocal of flood frequency, e.g., a 2 percent chance of occurrence is the reciprocal statement of a 50-year flood. (See Probability of Exceedance.)

Floodplain. Normally dry land areas subject to periodic temporary inundation by stream flow or tidal overflow. Land formed by deposition of sediment by water; alluvial land.

Floodplain Encroachment. An action within the limits of the base floodplain.

Flood Plane. The position occupied by the water surface of a stream during a particular flood. Also, loosely, the elevation of the water surface at various points along the stream during a particular flood.

Floodproof. To design and construct individual buildings, facilities, and their sites to protect against structural failure, to keep water out or reduce the effects of water entry.

Flood Stage. The elevation at which overflow of the natural banks of a stream begins to cause damage in the reach in which the elevation is measured. The elevation of the lowest bank of the reach. The term "lowest bank" is, however, not to be taken to mean an unusually low place or break in the natural bank through which the water inundates an unimportant and small area.

Flood Waters. Former stream waters which have escaped from a watercourse (and its overflow channel) and flow or stand over adjoining lands. They remain as such until they disappear from the surface by infiltration, evaporation, or return to a natural watercourse. They do not become surface waters by mingling with such waters, nor stream waters by eroding a temporary channel.

Flow. A term used to define the movement of water, silt, sand, etc.; discharge; total quantity carried by a stream.

Flow Line. A term used to describe the line connecting the low points in a watercourse.

Flow Regime. The system or order characteristic of streamflow with respect to velocity, depth, and specific energy.

Flow, steady. Flow at constant discharge.

Flow, unsteady. Flow on rising or falling stages.

Flow, varied. Flow in a channel with variable section.

Foreshore. The part of the shore lying between the ordinary high water mark or upper limit of wave wash traversed by the runup and return of waves and the water's edge at the low water.

Freeboard. (1) The vertical distance between the water surface elevation usually corresponding to the design flow and a point of interest such as a bridge beam, levee top or specific location on the roadway grade. (2) The distance between the normal operating level and the top of the sides of an open conduit; the crest of a dam, etc., designed to allow for wave action, superelevation, floating debris, or any other condition or emergency, without overtopping the structure. Freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccounted factors such as the accumulation of silt, trash, or aquatic growth in the channel; unforeseen embankment settlement, erratic hydrologic phenomena and variation of resistance or other coefficients from those assumed in design.

Free Outlet. A condition under which water discharges with no interference such as a pipe discharging into open air.

Free Water. Water which can move through the soil by force of gravity.

French Drain. A trench loosely backfilled with stones, the largest stones being placed in the bottom with the size of stones decreasing towards the top. The interstices between the stones serve as a passageway for water.

Friction. Energy-dissipating conflict among turbulent water particles disturbed by irregularities of channel surface.

Froude Number. A dimensionless expression of the ratio of inertia forces to gravity forces, used as an index to characterize the type of flow in a hydraulic structure in which gravity is the force producing motion and inertia is the resisting force. It is equal to a characteristic flow velocity (mean, surface, or maximum) of the system divided by the square root of the product of a characteristic dimension (as diameter of depth) and the gravity constant (acceleration due to gravity) all expressed in consistent units. $F_r = V/(gy)^{1/2}$

Gabion. A wire basket or cage filled with stone and placed as, or as part of, a bank-protection structure.

Gaging Station. A location on a stream where measurements of stage or discharge are customarily made. The location includes a reach of channel through which the flow is uniform, a control downstream from this reach and usually a small building to house the recording instruments.

Gorge. A narrow deep valley with steep or vertical banks.

Grade. Elevation of bed or invert of a channel.

Grade to Drain. A construction note often inserted on a plan for the purpose of directing the Contractor to slope a certain area in a specific direction, so that the surface waters will flow to a designated location.

Gradient (Slope). The rate of ascent or descent expressed as a percent or as a decimal as determined by the ratio of the change in elevation to the length.

Gradually Varied Flow. In this type of flow, changes in depth and velocity take place slowly over large distances, resistance to flow dominates and acceleration forces are neglected.

Grate. A framework of bars, usually cast iron or welded steel, used as a screen to cover the intake of a drainage inlet. See Standards Plans and Standard Specifications for requirements.

Ground Water. That water which is present under the earth's surface. Ground water is that situated below the surface of the land, irrespective of its source and transient status. Subterranean streams are flows of ground waters parallel to and adjoining stream waters, and usually determined to be integral parts of the visible streams.

Grouted. Bonded together with an inlay or overlay of cement mortar.

Guide Bank. An appendage to the highway embankment at or near a bridge abutment to guide the stream through the bridge opening.

Gulch. A relatively young, well-defined and sharply cut erosional channel.

Gully. Diminutive of gulch.

Head. Represents an available force equivalent to a certain depth of water. This is the motivating force in effecting the movement of water. The height of water above any point or plane of reference. Used also in various compound expressions, such as energy head, entrance head, friction head, static head, pressure head, lost head, etc.

Headcutting. Progressive scouring and degrading of a streambed at a relatively rapid rate in the upstream direction, usually characterized by one or a series of vertical falls.

High Water. Maximum flood stage of stream or lake; periodic crest stage of tide. Historic HW is stage recorded or otherwise known.

Hydraulic. Pertaining to water in motion and the mechanics of the motion.

Hydraulic Gradient. A line which represents the relative force available due to the potential energy available. This is a combination of energy due to the height of the water and the internal pressure. In any open channel, this line corresponds to the water surface. In a closed conduit, if several openings were placed along the top of the pipe and open tubes inserted, a line connecting the water surface in each of these tubes would represent the hydraulic grade line.

Hydraulic Jump (or Jump). Transition of flow from the rapid to the tranquil state. A varied flow phenomenon producing a rise in elevation of water surface. A sudden transition from supercritical flow to the complementary subcritical flow, conserving momentum and dissipating energy.

Hydraulic Mean Depth. The area of the flow cross section divided by the water surface width.

Hydraulic Radius. The cross sectional area of a stream of water divided by the length of that part of its periphery in contact with its containing conduit; the ratio of area to wetted perimeter.

Hydrograph. A graph showing stage, flow, velocity, or other property of water with respect to time.

Hydrographic. Pertaining to the measurement or study of bodies of water and associated terrain.

Hydrography. Water Surveys. The art of measuring, recording, and analyzing the flow of water; and of measuring and mapping watercourses, shore lines, and navigable waters.

Hydrologic. Pertaining to the cyclic phenomena of waters of the earth; successively as precipitation, runoff, storage and evaporation, and quantitatively as to distribution and concentration.

Hydrology. The science dealing with the occurrence and movement of water upon and beneath the land areas of the earth. Overlaps and includes portions of other sciences such as meteorology and geology. The particular branch of Hydrology that a design engineer is generally interested in is surface runoff which is the result of excess precipitation.

Hydrostatic. Pertaining to pressure by and within water due to gravitation acting thru depth.

Hyetograph. Graphical representation of rainfall intensity against time.

Impinge. To strike and attack directly, as in curvilinear flow where the current does not follow the curve but continues on tangent into the bank on the outside of bend in the channel.

Incised Channel. Those channels which have been cut relatively deep into underlying formations by natural processes. Characteristics include relatively straight alignment and high, steep banks such that overflow rarely occurs, if ever.

Infiltration. The passage of water through the soil surface into the ground.

Inlet Time. The time required for storm runoff to flow from the most remote point, in flow time, of a drainage area to the point where it enters a drain or culvert.

Inlet Transition. A specially shaped entrance to a box or pipe culvert. It is shaped in such a manner that in passing from one flow condition to another, the minimum turbulence or interference with flow is permitted.

Inundate. To cover with a flood.

Invert. The bottom of a drainage facility along which the lowest flows would pass.

Invert Paving. Generally applies to metal pipes where it is desirable to improve flow characteristics or prevent corrosion at low flows. The bottom portion of the pipe is paved with an asphaltic material, concrete, or air-blown mortar.

Inverted Siphon. A pipe for conducting water beneath a depressed place. A true inverted siphon is a culvert which has the middle portion at a lower elevation than either the inlet or the outlet and in which a vacuum is created at some point in the pipe. A sag culvert is similar, but the vacuum is not essential to its operation.

Isohyetal Line. A line drawn on a map or chart joining points that receive the same amount of precipitation.

Isohyetal Map. A map containing isohyetal lines and showing rainfall intensities.

Isovel. Line on a diagram of a channel connecting points of equal velocity.

Jack (or Jack Straw). Bank protection element consisting of wire or cable strung on three mutually perpendicular struts connected at their centers.

Jacking Operations. A means of constructing a pipeline under a highway without open excavation. A cutting edge is placed on the first section of pipe and the pipe is forced ahead by hydraulic jacks. As the leading edge pushes ahead, the material inside the pipe is dug out and transported outside the pipe for disposal.

Jam. Wedged collection of drift in a constriction of a channel, such as a gorge or a bridge opening.

Jet. An effluent stream from a restricted channel, including a fast current through a slower stream.

Jetty. An elongated, artificial obstruction projecting into a stream or the sea from bank or shore to control shoaling and scour by deflection of strength of currents and waves.

Jump. Sudden transition from supercritical flow to the complementary subcritical flow, conserving momentum and dissipating energy; the hydraulic jump.

Kolk. Rotational flow about a horizontal axis, induced by a reef and breaking the surface in a boil.

Lake. A water filled basin with restricted or no outlet. Includes reservoirs, tidal ponds and playas.

Lag. Variously defined as time from beginning (or center of mass) of rainfall to peak (or center of mass) of runoff.

Laminar Flow. That type of flow in which each particle moves in a direction parallel to every other particle and in which the head loss is approximately proportional to the velocity (as opposed to turbulent flow).

Lateral. In a roadway drainage system, a drainage conduit transporting water from inlet points to the main drain trunk line.

Levee. An embankment on or along the bank of a stream or lake to protect outer lowlands from inundation. (See Dike)

Lining. Protective cover of the perimeter of a channel.

Littoral. Pertaining to or along the shore, particularly to describe currents, deposits, and drift.

Littoral Drift. The sedimentary material (sand) moved along the shoreline under the influence of waves and currents.

Littoral Transport. The movement of littoral drift along the shoreline by waves and currents. Includes movement parallel (longshore transport) and perpendicular (on-offshore transport) to the shore.

Local Depression. A low area in the pavement or in the gutter established for the special purpose of collecting surface waters on a street and directing these waters into a drainage inlet.

Longshore. Parallel to and near the shoreline.

Marginal. Within a borderland area; more general and extensive than riparian.

Marsh. An area of soft, wet, or periodically submerged land, generally treeless and usually characterized by grasses and other low vegetation.

Mature. Classification for streams which have established flat gradients not subject to further scour.

Maximum Historical Flood. The maximum flood that has been recorded or experienced at any particular highway location.

Mean Annual Flood. The flood discharge with a recurrence interval of 2.33 years.

Mean Depth. For a stream at any stage, the wetted normal section divided by the surface width. Hydraulic mean depth.

Meander. In connection with streams, a winding channel usually in an erodible, alluvial valley. A reverse or S-shaped curve or series of curves formed by erosion of the concave bank, especially at the downstream end, characterized by curved flow and alternating shoals and bank erosions. Meandering is a stage in the migratory movement of the channel, as a whole, down the valley.

Meander Plug (Clay Plug). Deposits of cohesive materials in old channel bendways. These plugs are sufficiently resistant to erosion to serve as essentially semi-permanent geological controls to advancing channel migrations.

Meander Scroll. Evidence of historical meander patterns in the form of lines visible on the inside of meander bends (particularly on aerial photographs) which resemble a spiral or convoluted form in ornamental design. These lines are concentric and regular forms in high sinuosity channels and are largely absent in poorly developed braided channels.

Mesh. Woven wire or other filaments used alone as revetment, or as retainer or container of masses of gravel or cobble.

Mud Flow. A well-mixed mass of water and alluvium which, because of its high viscosity, and low fluidity as compared with water, moves at a much slower rate, usually piling up and spreading out like a sheet of wet mortar or concrete.

Natural and Beneficial Floodplain Values. Includes but are not limited to fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, aquaculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.

Natural Channel Capacity. The maximum rate of flow in cubic feet per second that can pass through a channel without overflowing the banks

Navigable Waters. Those stream waters lawfully declared or actually used as such. Navigable Waters of the State of California are those declared by Statute. Navigable Waters of the United States are those determined by the Corps of Engineers or the U.S. Coast Guard to be so used in interstate or international commerce. Other streams have been held navigable by courts under the common law that navigability in fact is navigability in law.

Negative Projecting Conduits. A structure installed in a trench with the top below the top of trench, then covered with backfill and embankment. See Positive Projecting Conduit

Nonuniform Flow. A flow in which the velocities vary from point to point along the stream or conduit, due to variations in cross section, slope, etc.

Normal Depth. The depth at which flow is steady and hydraulic characteristics are uniform.

Normal Water Surface (Natural Water Surface). The free surface associated with flow in natural streams.

"n" Value. The roughness coefficient in the Manning formula for determination of the discharge coefficient in the Chezy formula,

$$V = C(RS)^{1/2}, \text{ where } C = \left(\frac{1.49}{n}\right)R^{1/6}$$

Nourishment. The process of replenishing a beach. It may be brought about naturally, by accretion due to the longshore transport, or artificially, by the deposition of dredged materials.

Off-Site Drainage. The handling of that water which originates outside the highway right of way.

On-Site Drainage. The handling of that water which originates inside the highway right of way.

Open Channel. Any conveyance in which water flows with a free surface.

Ordinary High Water Mark. The line on the shore established by the fluctuation of water and physically indicated on the bank (1.5 $\pm$ years return period)

Outfall. Discharge or point of discharge of a culvert or other closed conduit.

Outwash. Debris transported from a restricted channel to an unrestricted area where it is deposited to form an alluvial or debris cone or fan.

Overflow. Discharge of a stream outside its banks; the parallel channels carrying such discharge.

Overtopping Flood. The flood described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief.

Peak Flow. Maximum momentary stage or discharge of a stream in flood. Design Discharge.

Pebble. Stone 0.5 inch to 3-inch in diameter, including coarse gravel and small cobble.

Perched Water. Ground water located above the level of the water table and separated from it by a zone of impermeable material.

Percolating Waters. Waters which have infiltrated the surface of the land and move slowly downward and outward through devious channels (aquifers) unrelated to stream waters, until they reach an underground lake or regain and spring from the land surface at a lower point.

Permeability. The property of soils which permits the passage of any fluid. Permeability depends on grain size, void ratio, shape and arrangement of pores.

Permeable. Open to the passage of fluids, as for (1) pervious soils and (2) bank-protection structures.

Physiographic Region. A geographic area whose pattern of landforms differ significantly from that of adjacent regions.

Pier. Vertical support of a structure standing in a stream or other body of water. Used in a general sense to include bents and abutments.

Pile. A long, heavy timber or section of concrete or metal that is driven or jetted into the earth or bottom of a water body to serve as a structural support or protection.

Piping. The action of water passing through or under an embankment and carrying some of the finer material with it to the surface at the downstream face.

Plunge. Flow with a strong downward component, as in outfall drops, overbank falls, and surf attack on a beach.

Point of Concentration. That point at which the water flowing from a given drainage area concentrates. With reference to a highway, this would generally be either a culvert entrance or some point in a roadway drainage system.

Poised Stream. A term used by river engineers applying to a stream that over a period of time is neither degrading or aggrading its channel, and is nearly in equilibrium as to sediment transport and supply.

Positive Projecting Conduit. A structure installed in shallow trench with the top of the conduit projecting above the top of the trench and then covered with embankment. See Negative Projecting Conduit.

Potamology. The hydrology of streams.

Practicable. Capable of being done within reasonable natural, social, and economic constraints.

Precipitation. Discharge of atmospheric moisture as rain, snow or hail, measured in depth of fall or in terms of intensity of fall in unit time.

Prescriptive Rights. The operation of the law whereby rights may be established by long exercise of their corresponding powers or extinguished by prolonged failure to exercise such powers.

Preserve. To avoid modification to the functions of the natural floodplain environment or to maintain it, as closely as practicable, in its natural state.

Probability. The chance of occurrence or recurrence of a specified event within a unit of time, commonly expressed in 3 ways. Thus a 10-year flood has a chance of 0.1 per year and is also called a 10 percent-chance flood.

Probability of Exceedance. The statistical probability, expressed as a percentage, of a hydrologic event occurring or being exceeded in any given year. The probability (p) of a storm or flood is the reciprocal of the average recurrence interval (N).

Probable Maximum Flood. The flood discharge that may be expected from the most severe combination of critical meteorological and hydrological conditions that are reasonably possible in the region.

Pumping Plant. A complete pumping installation including a storage box, pump or pumps, standby pumps, connecting pipes, electrical equipment, pumphouse and outlet chamber.

Rack. An open upright structure, such as a debris rack.

Rainfall. Point Precipitation: That which registers at a single gauge. Area Precipitation: Adjusted point rainfall for area size.

Rainwash. The creep of soil lubricated by rain.

Range. Difference between extremes, as for stream or tide stage.

Rapidly Varied Flow. In this type of flow, changes in depth and velocity take place over short distances, acceleration forces dominate, and energy loss due to friction is minor.

Rapids. Swift turbulent flow in a rough steep reach.

Reach. The length of a channel uniform with respect to discharge, depth, area, and slope. More generally, any length of a river or drainage course.

Recession. Retreat of shore or bank by progressive erosion.

Reef. Generally, any solid projection from the bed of a stream or other body of water.

Regime. The system or order characteristic of a stream; its behavior with respect to velocity and volume, form of and changes in channel, capacity to transport sediment, amount of material supplied for transportation, etc.

Regimen. The characteristic behavior of a stream during ordinary cycles of flow.

Regulatory Floodway. The open floodplain area that is reserved in by Federal, State, or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount (not to exceed 1 foot as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program (NFIP)).

Reliction. Pertaining to being left behind. For example: that area of land is left behind by reliction when the water surface of a lake is lowered.

Repose. The stable slope of a bank or embankment, expressed as an angle or the ratio of horizontal to vertical projection.

Restore. To reestablish a setting or environment in which the functions of the natural and beneficial floodplain values adversely impacted by the highway agency can continue to operate.

Restriction. Artificial or natural control against widening of a channel, with or without construction.

Retard. Bank-protection structure designed to check the riparian velocity and induce silting or accretion.

Retarding Basin. Either a natural or man made basin with the specific function of delaying the flow of water from one point to another. This tends to increase the time that it takes all the water falling on the extremities of the drainage basin to reach a common point, resulting in a reduced peak flow at that point.

Retention Storage. Water which accumulates and ponds in natural or excavated depressions in the soil surface with no possibility for escape as runoff. (See Detention Storage)

Retrogression. Reversal of stream grading; i.e., aggradation after degradation, or vice versa.

Revetment. Bank protection to prevent erosion.

Riparian. Pertaining to the banks of a stream.

Riprap. A layer, facing, or protective mound of rubble or stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also, the stone used for this purpose.

Ripple. (1) The light fretting or ruffling of a water caused by a breeze. (2) Undulating ridges and furrows, or crests and troughs formed by action of the flow.

Risk. The consequences associated with the probability of flooding attributable to an encroachment. It includes the potential for property loss and hazard to life during the service life of the highway.

Risk Analysis. An economic comparison of design alternatives using expected total costs (construction costs plus risk costs) to determine the alternative with the least expected cost to the public. It must include probable flood-related costs during the service life of the facility for highway operation, maintenance, and repair, for highway aggravated flood damage to other property, and for additional or interrupted highway travel.

Riser. In mountainous terrain where much debris is encountered, the entrance to a culvert sometimes becomes easily clogged. Therefore, a corrugated metal pipe or a structure made of timber or concrete with small perforations, called a riser, is installed vertically to permit entry of water and prohibit the entry of mud and debris. The riser may be increased in height as the need occurs.

River. A large stream, usually active when any streams are flowing in the region.

Rock. (1) Cobble, boulder or quarry stone as a construction material. (2) Hard natural mineral, in formation as in piles of talus.

Rounded Inlet. The edges of a culvert entrance that are rounded for smooth transition which reduces turbulence and increases capacity.

RSP Fabric. (See Filter Fabric).

Rubble. Rough, irregular fragments of rock or concrete.

Runoff. (1) The surface waters that exceed the soil's infiltration rate and depression storage. (2) The portion of precipitation that appears as flow in streams. Drainage or flood discharge which leaves an area as surface flow or a pipeline flow, having reached a channel or pipeline by either surface or subsurface routes.

Runup. The rush of water up a beach or structure, associated with the breaking of a wave. The amount of runup is measured according to the vertical height above still water level that the rush of water reaches.

Sag Culvert (or Sag Pipe). A pipeline with a dip in its grade line crossing over a depression or under a highway, railroad, canal, etc. The term inverted siphon is common but inappropriate as no siphonic action is involved. The term "sag pipe" is suggested as a substitute.

Sand. Granular soil coarser than silt and finer than gravel, ranging in diameter from 0.002 inch to 0.2 inch.

Scour. The result of erosive action of running water, primarily in streams, excavating and carrying away material from the bed and banks. Wearing away by abrasive action.

Scour, General. The removal of material from the bed and banks across all or most of the width of a channel, as a result of a flow contraction which causes increased velocities and bed shear stress.

Scour, Local. Removal of material from the channel bed or banks which is restricted to a minor part of the width of a channel. This scour occurs around piers and embankments and is caused by the actions of vortex systems induced by the obstruction to the flow.

Scour, Natural. Removal of material from the channel bed or banks which occurs in streams with the migration of bed forms, shifting of the thalweg and at bends and natural contractions.

Sea. Ocean or other body of water larger than a lake; state of agitation of any large body of water.

Seawall. A structure separating land and water areas, primarily designed to prevent erosion and other damage due to wave action. (See bulkhead).

Sediment. Fragmentary material that originates from weathering of rocks and is transported by, suspended in, or deposited by water.

Sedimentation. Gravitational deposit of transported material in flowing or standing water.

Seepage. Percolation of underground water thru the banks and into a stream or other body of water.

Seiche. A standing wave oscillation of an enclosed waterbody that continues, pendulum fashion, after the cessation of the originating force, which may have been either seismic or atmospheric.

Seismic Wave. A gravity wave caused by an earthquake.

Sheet Flow. Any flow spread out and not confined; i.e., flow across a flat open field.

Sheet Pile. A pile with a generally slender, flat cross-section that is driven into ground or bottom of a water body and meshed or interlocked with like members to form a wall or bulkhead.

Shoal. A shallow region in flowing or standing water, especially if made shallow by deposition.

Shoaling. Deposition of alluvial material resulting in areas with relatively shallow depth.

Shore. The narrow strip of land in immediate contact with the water, including the zone between high and low water lines. See backshore, foreshore, onshore, offshore, longshore, and nearshore.

Significant Encroachment. A highway encroachment and any direct support of likely base floodplain development that would involve one or more of the following construction or flood related impacts:

- A significant potential for interruption or termination of a transportation facility which is needed for emergency vehicles or provides a community's only evacuation route.
- A significant risk, or
- A significant adverse impact on natural and beneficial floodplain values.

Silt. (1) *Water-Borne Sediment.* Detritus carried in suspension or deposited by flowing water, ranging in diameter from 0.0002 inch to 0.002 inch. The term is generally confined to fine earth, sand, or mud, but is sometimes both suspended and bedload. (2) *Deposits of Water-Borne Material.* As in a reservoir, on a delta, or on floodplains.

Sinuosity. The ratio of the length of the river thalweg to the length of the valley proper.

Skew. When a drainage structure is not normal (perpendicular) to the longitudinal axis of the highway, it is said to be on a skew. The skew angle is the smallest angle between the perpendicular and the axis of the structure.

Slide. Gravitational movement of an unstable mass of earth from its natural position.

Slipout. Gravitational movement of an unstable mass of earth from its constructed position. Applied to embankments and other man-made earthworks.

Slope. (1) Gradient of a stream. (2) Inclination of the face of an embankment, expressed as the ratio of horizontal to vertical projection; or (3) The face of an inclined embankment or cut slope. In hydraulics it is expressed as percent or in decimal form.

Slough. (1) Pronounced SLU. A side or overflow channel in which water is continually present. It is stagnant or slack; also a waterway in a tidal marsh. (2) Pronounced SLUFF. Slide or slipout of a thin mantle of earth, especially in a series of small movements.

Slugflow. Flow in culvert or drainage structure which alternates between full and partly full. Pulsating flow -- mixed water and air.

Soffit. The bottom of the top -- (1) With reference to a bridge, the low point on the underside of the suspended portion of the structure. (2) In a culvert, the uppermost point on the inside of the structure.

Specific Energy. The energy contained in a stream of water, expressed in terms of head, referred to the bed of a stream. It is equal to the mean depth of water plus the velocity head of the mean velocity.

Spur Dike. A structure or embankment projecting a short distance into a stream from the bank and at an angle to deflect flowing water away from critical areas.

Stage. The elevation of a water surface above its minimum; also above or below an established "low water" plane; hence above or below any datum of reference; gage height.

Standing Wave. The motion of swiftly flowing stream water, that resembles a wave, but is formed by decelerating or diverging flow that does not quite produce a hydraulic jump. A term which when used to describe the upper flow regime in alluvial channels, means a vertical oscillation of the water surface between fixed nodes without appreciable progression in either an upstream or downstream direction. To maintain the fixed position, the wave must have a celerity (velocity) equal to the approach velocity in the channel, but in the opposite direction.

Steady Flow. A flow in which the flow rate or quantity of fluid passing a given point per unit of time remains constant.

Stone. Rock or rock-like material; a particle of such material, in any size from pebble to the largest quarried blocks.

Storage. Detention, or retention of water for future flow, naturally in channel and marginal soils or artificially in reservoirs.

Storage Basin. Space for detention or retention of water for future flow, naturally in channel and marginal soils, or artificially in reservoirs.

Storm. A disturbance of the ordinary, average conditions of the atmosphere which, unless specifically qualified, may include any or all meteorological disturbances, such as wind, rain, snow, hail, or thunder.

Storm Drain. That portion of a drainage system expressly for collecting and conveying former surface water in an enclosed conduit. Often referred to as a "storm sewer", storm drains include inlet structures, conduit, junctions, manholes, outfalls and other appurtenances.

Storm Water Management. The recognition of adverse drainage resulting from altered runoff and the solutions resulting from the cooperative efforts of public agencies and the private sector to mitigate, abate, or reverse those adverse results.

Strand. (1) To lodge on bars, banks, or overflow plain, as for drift. (2) Bar of sediment connecting two regions of higher ground.

Stream. Water flowing in a channel or conduit, ranging in size from small creeks to large rivers.

Stream Power. An expression used in predicting bed forms and hence bed load transport in alluvial channels. It is the product of the mean velocity, the specific weight of the water-sediment mixture, the normal depth of flow and the slope.

Stream Response. Changes in the dynamic equilibrium of a stream by any one, or combination of various causes.

Stream Waters. Former surface waters which have entered and now flow in a well defined natural watercourse, together with other waters reaching the stream by direct precipitation or rising from springs in bed or banks of the watercourse. They continue as stream waters as long as they flow in the watercourse, including overflow and multiple channels as well as the ordinary or low-water channel.

Strutting. Elongation of the vertical axis of pipe prior to installing in a trench. After the backfill has been placed around the pipe and compacted, the wires or rods holding the pipe in its distorted shape are removed. Greater side support from the earth is developed when the pipe tends to return to its original shape. Generally used on pipes which because of size or thinness of the metal would tend to deform during construction operations. Arches are strutted diagonally per standard or special plan.

Subcritical Flow. In this state, gravity forces are dominant, so that the flow has a low velocity and is often described as tranquil and streaming. Also, that flow which has a Froude number less than one.

Subdrain. A conduit for collecting and disposing of underground water. It generally consists of a pipe, with perforations in the bottom through which water can enter.

Subsidence. General lowering of land surface by consolidation or removal of underlying soil.

Sump. In drainage, any low area which does not permit the escape of water by gravity flow.

Supercritical Flow. In this state, inertia forces are dominant, so that flow has a high velocity and is usually described as rapid, shooting and torrential. Also, that flow which has a Froude number greater than one.

Support Base Floodplain Development. To encourage, allow, serve, or otherwise facilitate additional base floodplain development. Direct support results from an encroachment, while indirect support results from an action out of the base floodplain.

Surf. The breaking of waves and swell on the foreshore and offshore shoals.

Surface Runoff. The movement of water on earth's surface, whether flow is over surface of ground or in channels.

Surface Waters. Surface waters are those which have been precipitated on the land from the sky or forced to the surface in springs, and which have then spread over the surface of the ground without being collected into a definite body or channel. They appear as puddles, sheet or overland flow, and rills, and continue to be surface waters until they disappear from the surface by infiltration or evaporation, or until by overland or vagrant flow they reach well-defined watercourses or standing bodies of water like lakes or seas.

Surge. A sudden swelling of discharge in unsteady flow.

Suspended Load. Sediment that is supported by the upward components of turbulent currents in a stream and that stay in suspension for appreciable amount of time.

Swale. A shallow, gentle depression in the earth's surface. This tends to collect the waters to some extent and is considered in a sense as a drainage course, although waters in a swale are not considered stream waters.

Swamp. An area of shallow pondage or saturated surface, the water being fresh or acidic and the area usually covered with rank vegetation.

Swell. Waves generated by a distant storm, usually regular and fully harmonic.

Talus. Loose rocks and debris disintegrated from a steep hill or cliff standing at repose along the toe.

Tapered Inlet. A transition to direct the flow of water into a channel or culvert. A smooth transition to increase hydraulic efficiency of an inlet structure.

Terrace. Berm or bench-like earth embankment, with a nearly level plain bounded by rising and falling slopes.

Tetrahedron. Bank protection element, basically composed of 6 steel or concrete struts joined like the edges of a triangular pyramid, together with subdividing struts and tie wires or cables.

Tetrapod. Bank protection element, precast of concrete, consisting of 4 legs joined at a central block, each leg making an angle of 109.5 degrees with the other three, like rays from the center of a tetrahedron to the center of each face.

Texture. Arrangement and interconnection of surface and near-surface particles of terrain or channel perimeter.

Thalweg. The line following the lowest part of a valley, whether under water or not. Usually the line following the deepest part of the bed or channel of a river.

Thread. The central element of a current, continuous along a stream.

Tide. The periodic rising and falling of the ocean and connecting bodies of water that results from gravitational attraction of the moon and sun acting on the rotating earth.

Time of Concentration. The time required for storm runoff to flow from the most remote point, in flow time, of a drainage area to the point under consideration. It is usually associated with the design storm.

Topping. The top layer on horizontal revetments or rock structures; also capping or cap stones.

Training. Control of direction of currents.

Transition. A relatively short reach or conduit leading from one waterway section to another of different width, shape, or slope.

Transport. To carry solid material in a stream in solution, suspension, saltation, or entrainment.

Trash Rack. A grid or screen across a stream designed to catch floating debris.

Trough. Space between wave crests and the water surface below it.

Trunk (or Trunk Line). In a roadway drainage system, the main conduit for transporting the storm waters. This main line is generally quite deep in the ground so that laterals coming from fairly long distances can drain by gravity into the trunk line.

Tsunami. A gravity wave caused by an underwater seismic disturbance (such as sudden faulting, landsliding or volcanic activity).

Turbulence. A state of flow wherein the water is agitated by cross-currents and eddies, as opposed to a condition of flow that is quiet and laminar.

Turbulent Flow. That type of flow in which any particle may move in any direction with respect to any other particle, and in which the head loss is approximately proportional to the square of the velocity.

Undercut. Erosion of the low part of a steep bank so as to compromise stability of the upper part.

Underflow. The downstream flow of water through the permeable deposits that underlie a stream. (1) Movement of water through a pervious subsurface stratum, the flow of percolating water; or water under ice, or under a structure. (2) The rate of flow or discharge of subsurface water.

Undertow. Current outward from a wave-swept shore carrying solid particles swept or scoured from the beach or foreshore.

Unsteady Flow. A flow in which the velocity changes with respect to space and time.

Updrift. The direction opposite that of the predominant movement of littoral materials.

Uplift. Upward hydrostatic pressure on base of an impervious structure.

Velocity. The rate of motion of objects or particles, or of a stream of particles.

Velocity Head. A term used in hydraulics to represent the kinetic energy of flowing water. This "head" is represented by a column of standing water equivalent in potential energy to the kinetic energy of the moving water calculated as ($V^2/2g$) where the "V" represents the velocity in feet per second and "g" represents the potential acceleration due to gravity, in feet per second per second.

Vernal Pools. Seasonally flooded landscape depressions that support distinctive (and many times rare) plant and animal species adapted to periodic or continuous inundation during the wet season, and the absence of either ponded water or wet soil during the dry season.

Wash. Floodplain or active channel of an ephemeral stream, usually in recent alluvium.

Watercourse. A definite channel with bed and banks within which water flows, either continuously or in season. A watercourse is continuous in the direction of flow and may extend laterally beyond the definite banks to include overflow channels contiguous to the ordinary channel. The term does not include artificial channels such as canals and drains, except natural channels trained or restrained by the works of man. Neither does it include depressions or swales through which surface or errant waters pass.

Watershed. The area that contributes surface water runoff into a tributary system or water course.

Water Table. The surface of the groundwater below which the void spaces are completely saturated.

Waterway. (1) That portion of a watercourse which is actually occupied by water (2) A navigable inland body of water.

Wave. (1) An oscillatory movement of water on or near the surface of standing water in which a succession of crests and troughs advance while particles of water follow cyclic paths without advancing. (2) Motion of water in a flowing stream so as to develop the surficial appearance of a wave.

Wave Height. The vertical distance between a wave crest and the preceding trough.

Wave Length. The horizontal distance between similar points on two successive waves (e.g., crest to crest or trough to trough), measured in the direction of wave travel.

Wave Period. The time in which a wave crest travels a distance equal to one wave length. Can be measured as the time for two successive wave crests to pass a fixed point.

Weephole. A hole in a wall, invert, apron, lining, or other solid structure to relieve the pressure of groundwater.

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

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

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

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

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

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

Topic 807 - Selected Drainage References

807.1 Introduction

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

807.2 Federal Highway Administration Hydraulic Publications

Copies of publications identified with an NTIS or GPO number may be ordered as follows:

NTIS - Send a check to:

National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650

GPO - Send a check to:

Superintendent of Documents
Government Printing Office
Washington, D.C. 20402
(202) 783-3238

(1) Hydraulic Engineering Circulars (HEC).

HEC No.	Title	Date	FHWA # NTIS #
9	Debris-Control Structures	2005	IF-04-016
14	Hydraulic Design of Energy Dissipators for Culverts and Channels	2006	NHI-06-086
15	Design of Roadside Channels with Flexible Linings	2005	IF-05-114
17	The Design of Encroachments on Flood Plains Using Risk Analysis	1981	EPD-86-112 PB86-182110/AS
18	Evaluating Scour at Bridges	2012	HIF-12-003
20	Stream Stability at Highway Structures	2012	HIF-12-004
21	Bridge Deck Drainage Systems	1993	SA-92-010 PB94-109584
22	Urban Drainage Design Manual	2009	NHI-10-009
23	Bridge Scour and Stream Instability Countermeasures	2009	NHI-09-111 NHI-09-012
24	Highway Stormwater Pump Station Design	2001	NHI-01-007
25	Highways in the Coastal Environment	2008	NHI-07-096
26	Culvert Designer Aquatic Organism Passage	2010	HIF-11-008

(2) Hydraulic Design Series (HDS).

HDS No.	Title	Date	FHWA # NTIS #
2	Highway Hydrology	2002	NHI-02-001
3	Design Charts for Open-Channel Flow	1961	EPD-86-102 PB86-179249/AS
4	Introduction to Highway Hydraulics	2008	NHI-08-090
5	Hydraulic Design of Highway Culverts (GPO 050-001-00298-1)	2012	HIF-12-026
6	River Engineering for Highway Encroachments	2001	NHI-01-004
7	Hydraulic Design for Safe Bridges	2012	HIF-12-018

(3) Implementation Publications.

Title	Date	FHWA # NTIS #
Structural Design Manual for Improved Inlets and Culverts	1983	IP-83-6 PB84-153485
Culvert Inspection Manual	1986	IP-86-2 PB87-151809

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

(1) Highway Drainage Guidelines

The Drainage Guidelines is a collection of the guides previously published as individual volumes. These are:

I - Hydraulic Considerations in Highway Planning and Location
II - Hydrology
III - Erosion and Sediment Control in Highway Construction
IV - Hydraulic Design of Culverts
V - The Legal Aspects of Highway Drainage
VI - Hydraulic Analysis and Design of Open Channels
VII - Hydraulic Analysis for the Location and Design of Bridges
VIII- Hydraulic Aspects in Restoration and Upgrading of Highways
IX - Storm Drain Systems
X - Evaluating Highway Effects on Surface Water Environments
XI - Highways along Coastal Zones and Lakeshores
XII - Stormwater Management
XIII - Hydraulics Engineer Training and Career Development
XIV - Culvert Inspection and Rehabilitation
XV - Guidelines for Selecting and Utilizing Hydraulics Engineering Consultants

The current edition may be purchased through AASHTO, 444 North Capitol St., N.W., Suite 225, Washington D.C. 20001.

(2) *AASHTO Model Drainage Manual*

The Model Drainage Manual (MDM) is a comprehensive document covering a wide variety of transportation related hydraulic design issues. Developed for use by Federal, State, and local agencies, the MDM is a practice oriented document that allows the user agency to adopt the recommended values shown in the manual, or insert their own specific design policies and procedures.

807.4 California Department of Transportation

The following publications are available from the Caltrans Publications Unit, 1900 Royal Oaks Dr., Sacramento, CA 95815. Information on ordering and price can be checked by calling (916) 445-3520.

- Bridge Design Practice Manual
- Manual of Test - Volumes 1, 2, and 3
- Standard Plans
- Standard Specifications

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

- Magnitude and Frequency of Floods in California - Water Resources Investigation 77-21.
- Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States - Water-Supply Paper 2433.
- Guide For Determining Flood Flow Frequency - Bulletin #17B.
- Water Resources Data for California, Part 1, Volumes 1 and 2.
- Rock Riprap Design for Protection of Stream Channels Near Highway Structures (1987) Volumes 1 and 2 (1987).
- Regional Skew for California, and Flood Frequency for Selected Sites in the Sacramento-San Joaquin River Basin, Based on Data through Water Year 2006 - Scientific Investigations Report 2010-5260.

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

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

807.7 California Department of Water Resources

The California Department of Water Resources provides intensity, duration, and frequency data from the California Department of Water Resources network of rain gauges at the following website:
http://www.water.ca.gov/floodmgmt/hafoo/hb/csm/engineering/

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

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

807.9 U.S. Army Corps of Engineers

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

Topic 808 – Selected Computer Programs

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

Table 808.1

Summary of Related Computer Programs and Web Applications

	Storm Drains	Hydrology	Water Surface Profiles	Culverts	Roadside /Median Channels	Pavement Drainage	Pond Routing
FHWA Hydraulic Toolbox					x	x	
TR-55		x					
HEC-HMS (2)		x					x
HY-8				x			
HEC-RAS (1)			x				
FESWMS			x				
WMS		x		x			x
NOAA Atlas 14		x					
USGS StreamStats		x					
AutoDesk Civil 3D/Hydraflow	x	x				x	x

NOTES:

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

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

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

CHAPTER 810
HYDROLOGY

Topic 811 - General

Index 811.1 - Introduction

Hydrology is often defined as: "A science dealing with the properties, distribution, and circulation of water on the surface of the land, in the soil and underlying rocks, and in the atmosphere." This is a very broad definition encompassing many disciplines relating to water. The highway engineer is principally concerned with surface hydrology and controlling surface runoff. Controlling runoff includes the hydraulic design of drainage features for both cross highway drainage (Chapter 820) and removal of runoff from the roadway (Chapter 830).

The runoff of water over land has long been studied and some rather sophisticated theories and methods have been proposed and developed for estimating flood flows. Most attempts to describe the process have been only partially successful at best. This is due to the complexity of the process and interactive factors. The random nature of rainfall, snowmelt, and other sources of water further complicate the process.

It should be understood that there are no exact methods for hydrologic analysis. Different methods that are commonly used may produce significantly different results for a specific site and particular situation.

Although hydrology is not an exact science, it is possible to obtain solutions which are functionally acceptable to form the basis for design of highway drainage facilities.

More complete information on the principles and engineering techniques pertaining to hydrology for transportation and highway engineers may be found in FHWA Hydraulic Design Series (HDS) No. 2, Hydrology. Key aspects of hydrologic information and a general overview of hydrology relevant to highway engineering are more fully discussed in the AASHTO Highway Drainage Guidelines and the AASHTO Model Drainage Manual. Both of these publications cite appropriate and recommended references on specific aspects of hydrologic studies and research available to the highway design engineer requiring more thorough information on hydrologic analysis.

811.2 Objectives of Hydrologic Analysis

Regardless of the size or cost of the drainage feature the most important step prior to hydraulic design is estimating the discharge (rate of runoff) or volume of runoff that the drainage facility will be required to convey or control.

While some hydrologic analysis is necessary in establishing the quantity of surface water that must be considered in the design of all highway drainage facilities, the extent of such studies are to be commensurate with the importance of the highway, the potential for damage to the highway, loss of property, and hazard to life associated with the facilities.

The choice of analytical method must be a conscious decision made as each problem arises. To make an informed decision, the highway engineer must determine:

- What level of hydrologic analysis is justified.
- What data are available or must be collected.
- What methods of analysis are available including the relative strengths and weaknesses in terms of cost and accuracy.

Cross drainage design, Chapter 820, normally requires more extensive hydrologic analysis than is necessary for roadway drainage design, Chapter 830. The well known and relatively simple "Rational Method" (see Index 819.2) is generally adequate for estimating the rate or volume of runoff for the design of on-site roadway drainage facilities and removal of runoff from highway pavements.

811.3 Peak Discharge

Peak discharge is the maximum rate of flow of water passing a given point during or after a rainfall event. Peak discharge, often called peak flow, occurs at the momentary "peak" of the stream's flood hydrograph. (See Index 816.5, Flood Hydrograph.)

Design discharge, expressed as the quantity (Q) of flow in cubic feet per second (CFS), is the peak discharge that a highway drainage structure is sized to handle. Peak discharge is different for every storm and it is the highway engineer's responsibility to size drainage facilities and structures for the magnitude of the design storm and flood severity. The magnitude of peak discharge varies with the severity of flood events which is based on probability of exceedance (see Index 811.4). The selection of design storm frequency and flood probability are more fully discussed under Topic 818, Flood Probability and Frequency.

811.4 Flood Severity

Flood severity is usually stated in terms of:

- Probability of Exceedance, or
- Frequency of Recurrence.

Modern concepts tend to define a flood in terms of probability. Probability of exceedance, the statistical odds or chance of a flood of given magnitude being exceeded in any year, is generally expressed as a percentage. Frequency of recurrence is expressed in years, on the average, that a flood of given magnitude would be predicted. Refer to Topic 818 for further discussion of flood probability and frequency.

811.5 Factors Affecting Runoff

The highway engineer should become familiar with the many factors or characteristics that affect runoff before making a hydrologic analysis. The effects of many of the factors known to influence surface runoff only exist in empirical form. Extensive field data, empirically determined coefficients, sound judgment, and experience are required for a quantitative analysis of these factors. Relating flood flows to these causative factors has not yet advanced to a level of precise mathematical expression.

Some of the more significant factors which affect the hydraulic character of surface water runoff are categorized and briefly discussed in Topics 812 through 814. It is important to recognize that the factors discussed may exist concurrently within a watershed and their combined effects are very difficult to quantify.

Topic 812 - Basin Characteristics

812.1 Size

The size (area) of a drainage basin is the most important watershed characteristic affecting runoff. Determining the size of the drainage area that contributes to flow at the site of the drainage structure is a basic step in a hydrologic analysis regardless of the method used to evaluate flood flows. The drainage area, expressed in hectares or square miles, is frequently determined from field surveys, topographic maps, or aerial photographs.

812.2 Shape

The shape, or outline formed by the basin boundaries, affects the rate at which water is supplied to the main stream as it proceeds along its course from the runoff source to the site of the drainage structure. Long narrow watersheds generally give lower peak discharges than do fan or pear shaped basins.

812.3 Slope

The slope of a drainage basin is one of the major factors affecting the time of overland flow and concentration of rainfall (see Index 816.6, Time of Concentration). Steep slopes tend to result in shorter response time and increase the discharge while flat slopes tend to result in longer response time and reduce the discharge.

812.4 Land Use

Changes in land use nearly always cause increases in surface water runoff. Of all the land use changes, urbanization is the most dominant factor affecting the hydrology of an area.

Land use studies may be necessary to define present and future conditions with regard to urbanization or other changes expected to take place within the drainage basin.

Valuable information concerning land use trends is available from many sources such as:

- State, regional or municipal planning organizations.
- U.S. Geological Survey.

- U.S. Department of Agriculture (Water Branch - Natural Resource Economic Division.)

Within each District there are various organizations that collect, publish or record land use information. The District Hydraulics Engineer should be familiar with these organizations and the types of information they have available.

A criterion of good drainage design is that future development and land use changes which can reasonably be anticipated to occur during the design life of the drainage facility be considered in the hydraulic analysis and estimation of design discharge.

812.5 Soil and Geology

The type of surface soil which is characteristic of an area is an important consideration for any hydrologic analysis and is a basic input to the National Resources Conservation Service (NRCS) method. Rock formations underlying the surface soil and other geophysical characteristics such as volcanic, glacial, and river deposits can have a significant effect on run-off.

The major source of soil information is the National Resources Conservation Service (NRCS) of the U.S. Department of Agriculture. The address and telephone number of the NRCS office in California is:

2121 Second Street,
Building C
Davis, CA 95616-5475
(916) 757-8200

812.6 Storage

Interception and depression storage are generally not important considerations in highway drainage design and may be ignored in most hydrologic analysis. Interception storage is rainfall intercepted by vegetation and never becomes run-off. Depression storage is rainfall lost in filling small depressions in the ground surface, storage in transit (overland or channel flow), and storage in ponds, lakes or swamps.

Detention storage can have a significant effect in reducing the peak rate of discharge, but this is not always the case. There have been rare instances where artificial storage radically redistributes the discharges and higher peak discharges have resulted than would occur had the storage not been added.

The effect of flood-control reservoirs should be considered in evaluating downstream conditions, flood peaks, and river stages for design of highway structures. The controlling public agency or the owner should be contacted for helpful information on determining the effects, if any, on downstream highway drainage structures.

It is not uncommon for flood control projects to be authorized but never constructed because funds are not appropriated. Therefore a flood control project should exist or be under construction if its effects on a drainage system are to be considered.

812.7 Elevation

The mean elevation of a drainage basin and significant variations in elevation within a drainage basin may be important characteristics affecting run-off particularly with respect to precipitation falling as snow. Elevation is a basic input to some of the USGS Regional Regression Equations (see Index 819.2).

812.8 Orientation

The amount of runoff can be affected by the orientation of the basin. Where the general slope of the drainage basin is to the south it will receive more exposure to the heat of the sun than will a slope to the north. Such orientation affects transpiration, evaporation, and infiltration losses. Snowpack and the rate at which snow melts will also be affected. A basin's orientation with respect to the direction of storm movement can affect a flood peak. Storms moving upstream produce lower peaks than storms tending to move in the general direction of stream flow.

Topic 813 - Channel and Floodplain Characteristics

813.1 General

Streams are formed by the gathering together of surface waters into channels that are usually well defined. The natural or altered condition of the channels can materially affect the volume and rate of runoff and is a significant consideration in the hydrological analysis for cross drainage design.

A useful reference relative to problems associated with transverse and longitudinal highway encroachments upon river channels and floodplains is the FHWA Training and Design Manual, "Highways in the River Environment - Hydraulic and Environmental Design Considerations"

813.2 Length and Slope

The longer the channel the more time it takes for water to flow from the beginning of the channel to the site under consideration. Channel length and effective channel slope are important parameters in determining the response time of a watershed to precipitation events of given frequency.

In the case of a wide floodplain with a meandering main channel the effective channel length will be reduced during flood stages when the banks are overtopped and flow tends more toward a straight line.

813.3 Cross Section

Flood peaks may be estimated by using data from stream gaging stations and natural channel cross section information.

Although channel storage is usually ignored in the hydrologic analysis for the design of highway drainage structures, channel cross section may significantly affect discharge, particularly in wide floodplains with heavy vegetation.

If channel storage is considered to be a significant factor, the assistance of an expert in combining the analysis of basin hydrology and stream hydraulics should be sought. The U.S. Army Corps of Engineers has developed computer programs, HEC-1, HEC-HMS Flood Hydrograph Package and HEC-RAS, Water Surface Profiles, for this type of analysis. For modeling complex water surface profile problems, where one-dimensional models fail, FHWA has developed the Finite Element Surface Water Modeling System Two Dimensional Flow in a Horizontal Plane (FESWMS-2DH).

813.4 Hydraulic Roughness

Hydraulic roughness represents the resistance to flows in natural channels and floodplains. It affects both the time response of a drainage channel and channel storage characteristics. The lower the roughness, the higher the peak discharge and the shorter the time of the resulting hydrograph. The total volume of runoff however is virtually independent of hydraulic roughness.

Streamflow is frequently indirectly computed by using Manning's equation, see Index 866.3(4). Procedures for selecting an appropriate coefficient of hydraulic roughness, Manning's "n", may be found in the FHWA report, "Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains".

813.5 Natural and Man-made Constrictions

Natural constrictions, such as gravel bars, rock outcrops and debris jams as well as artificial constrictions such as diversion and storage dams, grade-control structures, and other water-use facilities may control or regulate flow. Their effect on the flood peak may be an important consideration in the hydrologic analysis.

813.6 Channel Modifications

Channel improvements such as channel-straightening, flood control levees, dredging, bank clearing and removal of obstructions tend to reduce natural attenuation and increase downstream flood peaks.

813.7 Aggradation - Degradation

Aggradation, deposited sediments, may lessen channel capacity and increase flood heights causing overflow at a lower discharge. Degradation, the lowering of the bed of a stream or channel, may increase channel capacity and result in a higher peak discharge.

The validity of hydrologic analysis using observed historical highwater marks may be affected by aggradation or degradation of the streambed. The effects of aggradation and degradation are important considerations in selecting an effective drainage system design to protect highways and adjacent properties from damage. For more information refer to the FHWA report entitled, "Stream Channel Degradation and Aggradation: Analysis of Impact to Highway Crossings".

813.8 Debris

The quantity and size of solid matter carried by a stream may affect the hydrologic analysis of a drainage basin. Bulking due to mud, suspended sediment and other debris transported by storm runoff may significantly increase the volume of flow, affect flow characteristics, and can be a major consideration in the hydraulic design of drainage structures. In particular, bulking factors are typically a consideration in determining design discharges for facilities with watersheds that are located within mountainous regions subject to fire and subsequent soil erosion, or in arid regions when the facility is in the vicinity of alluvial fans (see Index 872.3(5) for special considerations given to highways located across desert washes).

Debris control methods, structures, and design considerations are discussed in Topic 822, Debris Control.

The District Hydraulics Engineer should be consulted for any local studies that may be available. If both stream gage data and local studies are available, a determination of whether post-fire peak flows are included within the data record should be made. Consideration should be given to treating a significant post-fire peak as the design discharge in lieu of the peak discharge obtained through gage analysis for a given probability flood event. Records of stream discharge from burned and long-unburned (unburned for 40 years or more years) areas have showed peak discharge increases from 2 to 30 times in the first year after burning. In mountainous regions subject to fire with no local studies available, the U.S. Forest Service should be contacted for fire history in order to determine if there is a significant post-fire peak within the stream records.

Topic 814 - Meteorological Characteristics

814.1 General

Meteorology is the science dealing with the earth's atmosphere, especially the weather. As applied to hydrology for the highway designer the following elements of meteorological phenomena are considered the more important factors affecting runoff and flood predictions.

814.2 Rainfall

Rainfall is the most common factor used to predict design discharge. Unfortunately, due to the many interactive factors involved, the relationship between rainfall and runoff is not all that well defined. Intuitively, engineers know and studies confirm, that runoff increases in proportion to the rainfall on a drainage basin. Highway design engineers are cautioned about assuming that a given frequency storm always produces a flood of the same frequency. There are analytical techniques for ungaged watersheds that are based on this assumption. A statistical analysis of extensive past rainfall records should be made before such a correlation is accepted.

Rainfall event characteristics which are important to highway drainage design are:

- Intensity (rate of rainfall)
- Duration (time rainfall lasts)
- Frequency (statistical probability of how often rainfall will occur)
- Time Distribution (intensity hyetograph)
- Storm Type (orographic, convective or cyclonic)
- Storm Size (localized or broad areal extent)
- Storm Movement (direction of storm)

814.3 Snow

Much of the precipitation that falls in the mountainous areas of the state falls as frozen water in the form of snow, hail, and sleet. Since frozen precipitation cannot become part of the runoff until melting occurs it is stored as snowpack until thawed by warmer weather.

Rain upon an accumulation of snow can cause a much higher peak discharge than would occur from rainfall alone. The parameters of snow which may need to be considered in quantifying peak flood runoff are:

- Mean annual snowfall
- Water content of snowpack
- Snowmelt rate

814.4 Evapo-transpiration

Evaporation and transpiration are two natural processes by which water reaching the earth's surface is returned to the atmosphere as vapor. The losses due to both phenomena are important to long term hydrology and water balance in the watershed and are usually ignored in the hydrologic analysis for the design of highway drainage facilities.

814.5 Tides and Waves

The combined effect of upland runoff and tidal action is a primary consideration in the design of highway drainage structures and shore protection facilities along the coastlines, on estuaries, and in river delta systems.

The time and height of high and low water caused by the gravitational attraction of the sun and moon upon the earth's oceans are precisely predictable. Information on gravitational tides and tidal bench marks for the California Coastline is available from:

State Lands Commission
NOS Marine Boundary Program
1807 13th Street
Sacramento, CA 95814

Or from the following web-site:
http://co-ops.nos.noaa.gov/bench.html.

One of the most devastating forces affecting the coastline occurs when an astronomical high tide and a storm of hurricane proportion arrive on the land at the same time. This is also true of the effect of a tsunami. A tsunami is a wave caused by an earthquake at sea. If shore protection were designed to withstand the forces of a tsunami, it would be extremely costly to construct. Since it would be so costly and the probability of occurrence is so slight, such a design may not be justified.

Wind-waves directly affect coastal structures and cause dynamic changes in coastal morphology. The U.S. Corps of Engineers collects and publishes data which may be used to predict size of Pacific Coast wind-waves. Information pertaining to the California coastline from the Mexican border north to Cape San Martin can be obtained from:

U.S. Army Corps of Engineers
Los Angeles District
P.O. Box 2711
Los Angeles, CA 90053
(213) 688-5400

For information from Cape San Martin to the Oregon border from:

U.S. Army Corps of Engineers
San Francisco District
211 Main Street
San Francisco, CA 94105
(415) 556-3582

Wind-waves are also generated on large inland bodies of water and their effect should be considered in the design of shoreline highway facilities.

Topic 815 - Hydrologic Data

815.1 General

The purpose for which a hydrologic study is to be made will determine the type and amount of hydrologic data needed. The accuracy necessary for preliminary studies is usually not as critical as the desirable accuracy of a hydrologic analysis to be used for the final design of highway drainage structures. If data needs can be clearly identified, data collection and compilation efforts can be tailored to the importance of the project.

Data needs vary with the methods of hydrologic analysis. Highway engineers should remember that there is no single method applicable to all design problems. They should make use of whatever hydrologic data that has been developed by others whenever it is available and applicable to their needs.

Frequently there is little or no data available in the right form for the project location. For a few locations in the State, so much data has been compiled that it is difficult to manage, store, and retrieve the information that is applicable to the project site.

815.2 Categories

For most highway drainage design purposes there are three primary categories of hydrologic data:

(1) *Surface Water Runoff.* This includes daily and annual averages, peak discharges, instantaneous values, and highwater marks.

(2) *Precipitation.* Includes rainfall, snowfall, hail, and sleet.

(3) *Drainage Basin Characteristics.* Adequate information may not be readily available but can generally be estimated or measured from maps, field reviews or surveys. See Topic 812 for a discussion of basin characteristics.

Other special purpose categories of hydrologic data which may be important to specific problems associated with a highway project are:

- Sediment and debris transport
- Snowpack variations
- Groundwater levels and quantity
- Water quality

815.3 Sources

Hydrologic data necessary for the design of cross drainage (stream crossings) are usually obtained from a combination of sources.

(1) *Field Investigations.* A great deal of the essential information can only be obtained by visiting the site. Except for extremely simple designs or the most preliminary analysis, a field survey or site investigation should always be made.

To optimize the amount and quality of the hydrologic data collected the field survey should be well planned and conducted by an engineer with general knowledge of drainage design. Data collected are to be documented. When there is reason to believe that a potential for significant risks or impacts associated with the design of drainage facilities may exist, a written report with maps and photographs may be necessary. (See Topic 804 for Floodplain Encroachments.) Appended to HDS No. 2 is a checklist for drainage studies and reports which may be a useful guide in the conduct of hydrologic studies. Typical data collected in a field survey are:

- Highwater marks
- Performance and condition of existing drainage structures
- Stream alignment
- Stream stability and scour potential
- Land use and potential development
- Location and nature of physical and cultural features
- Vegetative cover
- Upstream constraints on headwater elevation
- Downstream constraints
- Debris potential

(2) *Federal Agencies.* The following agencies collect and disseminate stream flow data:

- Geological Survey (USGS)
- Corps of Engineers (COE)
- Bureau of Reclamation (USBR)
- Soil Conservation Service (SCS)
- Forest Service (USFS)
- Bureau of Land Management (BLM)
- Federal Emergency Management Agency (FEMA)
- Environmental Protection Agency (EPA)

The USGS is the primary federal agency charged with collecting and maintaining water related data. Stream-gaging station data and other water related information collected by the USGS is published in Water Supply Papers and through the USGS Office of Surface Water website.

(3) *State Agencies.* The primary state agency collecting stream-gaging and precipitation (rain-gage and snowfall) data is the California Department of Water Resources (DWR).

(4) *Local Agencies.* Entities such as cities, counties, flood control districts, or local improvement districts study local drainage conditions and are often a valuable source of hydrologic data.

(5) *Private Sector.* Water using industries or utilities, railroads and local consultants frequently have pertinent hydrologic records and studies available.

815.4 Stream Flow

Once surface runoff water enters into a stream, it becomes "stream flow". Stream flow is the only portion of the hydrologic cycle in which water is so confined as to make possible reasonably accurate measurements of the discharges or volumes involved. All other measurements in the hydrologic cycle are, at best, only inadequate samples of the whole.

The two most common types of stream flow data are:

- Gaging Stations - data generally based on recording gage station observations with detailed information about the stream channel cross section. Current meter measurements of transverse channel velocities are made to more accurately reflect stream flow rates.

- Historic - data based on observed high water mark and indirect stream flow measurements.

Stream flow data are usually available as mean daily flow or peak daily flow. Daily flow is a measurement of the rate of flow in cubic feet per second (CFS) for the 24-hour period from midnight to midnight.

"Paleoflood" (ancient flood) data has been found useful in extending stream gaging station records. (See Topic 817 for further discussion on measuring stream flow)

815.5 Rainfall

Rainfall data are collected by recording and non-recording rain gages. Rainfall collected by vertical cylindrical rain gages of about 8 inches in diameter is designated as "point rainfall".

Regardless of the care and precision used, rainfall measurements from rain gages have inherent and unavoidable shortcomings. Snow and wind problems frequently interrupt rainfall records. Extreme rainfall data from recording rain gage charts are generally underestimated.

Rain gage measurements are seldom used directly by highway engineers. The statistical analysis which must be done with precipitation measurements is nearly always performed by qualified hydrologists and meteorologists such as those employed by the Department of Water Resources (DWR). The intensity-duration-frequency (IDF) tables and curves are the products of rainfall measurement analyses which have direct application to highway drainage design.

815.6 Adequacy of Data

All hydrologic data that has been collected must be evaluated and compiled into a usable format. Experience, knowledge and judgment are an important part of data evaluation. It must be ascertained whether the data contains inconsistencies or other unexplained anomalies which might lead to erroneous calculations and conclusions that could result in the over design or under design of drainage structures.

Topic 816 - Runoff

816.1 General

The process of surface runoff begins when precipitation exceeds the requirements of:

- Vegetal interception.
- Infiltration into the soil.
- Filling surface depressions (puddles, swamps and ponds). As rain continues to fall, surface waters flow down slope toward an established channel or stream.

816.2 Overland Flow

Overland flow is surface waters which travel over the ground as sheet flow, in rivulets and in small channels to a watercourse.

816.3 Subsurface Flow

Waters which move laterally through the upper soil surface to streams are called "interflow" or "subsurface flow". For the purpose of highway drainage hydrology, where peak design discharge (flood peaks) are the primary interest, subsurface flows are considered to be insignificant. Subsurface flows travel slower than overland flow.

While groundwater and subsurface water may be ignored for runoff estimates, their detrimental effect upon highway structural section stability cannot be overstated. See Chapter 840, Subsurface Drainage.

816.4 Detention and Retention

Water which accumulates and ponds in low points or depressions in the soil surface with no possibility for escape as runoff is in retention storage. Where water is moving over the land it is in detention storage. Detained water, as opposed to retained water, contributes to runoff.

816.5 Flood Hydrograph and Flood Volume

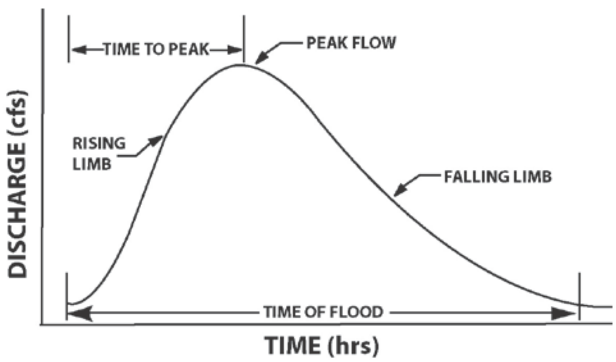
In response to a rainstorm the quantity of water flowing in a stream increases. The water level rises and may continue to do so after rainfall ceases. The response of an affected stream, during and after a storm event, can be pictured by plotting discharge against time to produce a flood hydrograph. The principal elements of a typical flood hydrograph are shown in Figure 816.5

Flood volume is the area under the flood hydrograph. Although flood volume is not normally a consideration in the design of highway drainage facilities, it is occasionally used in the hydrologic analysis for other design parameters.

Information on flood hydrographs and methods to estimate the hydrograph may be found in Chapters 6, 7 and 8 of HDS No. 2, Hydrology.

Figure 816.5

Typical Flood Hydrograph



816.6 Time of Concentration (Tc) and Travel Time (Tt)

Time of concentration is defined as the time required for storm runoff to travel from the hydraulically most remote point of the drainage basin to the point of interest.

An assumption made in some of the hydrologic methods for estimating peak discharge, such as the Rational and NRCS Methods (Index 819.2), is that maximum flow results when rainfall of uniform intensity falls over the entire watershed area and the duration of that rainfall is equal to the time of concentration. Time of concentration (T_C) is typically the cumulative sum of three travel times, including:

- Sheet flow
- Shallow concentrated flow
- Channel flow

For all-paved watersheds (e.g., parking lots, roadway travel lanes and shoulders, etc.) it is not necessary to calculate a separate shallow concentrated flow travel time segment. Such flows will typically transition directly from sheet flow to channel flow or be intercepted at inlets with either no, or inconsequential lengths of, shallow concentrated flow.

In many cases a minimum time of concentration will have to be assumed as extremely short travel times will lead to calculated rainfall intensities that are overly conservative for design purposes. For all-paved areas it is recommended that a minimum time of concentration of 5 minutes be used. For rural or undeveloped areas, it is recommended that a minimum T_C of 10 minutes be used for most situations. However, for slopes steeper than 1V:10H, or where there is limited opportunity for surface storage, a T_C of 5 minutes should be assumed.

Designers should be aware that maximum runoff estimates are not always obtained using rainfall intensities determined by the time of concentration for the total area. Peak runoff estimates may be obtained by applying higher rainfall intensities from storms of short duration over a portion of the watershed.

(1) *Sheet flow travel time.* Sheet flow is flow of uniform depth over plane surfaces and usually occurs for some distance after rain falls on the ground. The maximum flow depth is usually less than 0.8 inches - 1.2 inches. For unpaved areas, sheet flow normally exists for a distance less than 80 feet - 100 feet. An upper limit of 300 feet is recommended for paved areas.

A common method to estimate the travel time of sheet flow is based on kinematic wave theory and uses the Kinematic Wave Equation:

$$T_t = \frac{0.93 L^{3/5} n^{3/5}}{i^{2/5} S^{3/10}}$$

where

T_t = travel time in minutes.

L = Length of flow path in feet.

S = Slope of flow in feet per feet.

n = Manning's roughness coefficient for sheet flow (see Table 816.6A).

i = Design storm rainfall intensity in inches per hour.

If T_t is used (as part of T_C) to determine the intensity of the design storm from the IDF curves, application of the Kinematic Wave Equation becomes an iterative process: an assumed value of T_t is used to determine i from the IDF curve; then the equation is used to calculate a new value of T_t which in turn yields an updated i. The process is repeated until the calculated T_t is the same in two successive iterations.

To eliminate the iterations, use the following simplified form of the Manning's kinematic solution:

$$T_t = \frac{0.42 L^{4/5} n^{4/5}}{P_2^{1/2} s^{2/5}}$$

where P_2 is the 2-year, 24-hour rainfall depth in inches (ref. NOAA Atlas 14, http://hdsc.nws.noaa.gov/hdsc/pfds/).

The use of flow length alone as a limiting factor for the Kinematic wave equation can lead to circumstances where the underlying assumptions are no longer valid. Over prediction of travel time can occur for conditions with significant amounts of depression storage, where there is high Manning's n-values or for flat slopes. One study suggests that the upper limit of applicability of the Kinematic wave equation is a function of flow length, slope and Manning's roughness coefficient. This study used both field and laboratory data to propose an upper limit of 100 for the composite parameter of $nL/s^{1/2}$. It is recommended that this criteria be used as a check where the designer has uncertainty on the maximum flow length to which the Kinematic wave equation can be applied to project conditions.

Where sheet flow travel distance cannot be determined, a conservative alternative is to assume shallow concentrated flow conditions without an independent sheet flow travel time conditions. See Index 816.6(2).

Table 816.6A
Roughness Coefficients For Sheet Flow

Surface Description	***n***
Hot Mix Asphalt	0.011-0.016
Concrete	0.012-0.014
Brick with cement mortar	0.014
Cement rubble	0.024
Fallow (no residue)	0.05
Grass	
Short grass prairie	0.15
Dense grass	0.24
Bermuda Grass	0.41
Woods(1)	
Light underbrush	0.40
Dense underbrush	0.80

(1) Woods cover is considered up to a height of 1 inch, which is the maximum depth obstructing sheet flow.

(2) *Shallow concentrated flow travel time.* After short distances, sheet flow tends to concentrate in rills and gullies, or the depth exceeds the range where use of the Kinematic wave equation applies. At that point the flow becomes defined as shallow concentrated flow. The Upland Method is commonly used when calculating flow velocity for shallow concentrated flow. This method may also be used to calculate the total travel time for both the sheet flow and the shallow concentrated flow segments under certain conditions (e.g., where use of the Kinematic wave equation to predict sheet flow travel time is questionable, or where the designer cannot reasonably identify the point where sheet flow transitions to shallow concentrated flow).

Average velocities for the Upland Method can be taken directly from Figure 816.6 or may be calculated from the following equation:

$$V = (3.28)\, kS^{1/2}$$

Where S is the slope in percent and k is an intercept coefficient depending on land cover as shown in Table 816.6B.

Figure 816.6

Velocities for Upland Method of Estimating Travel Time for Shallow Concentrated Flow

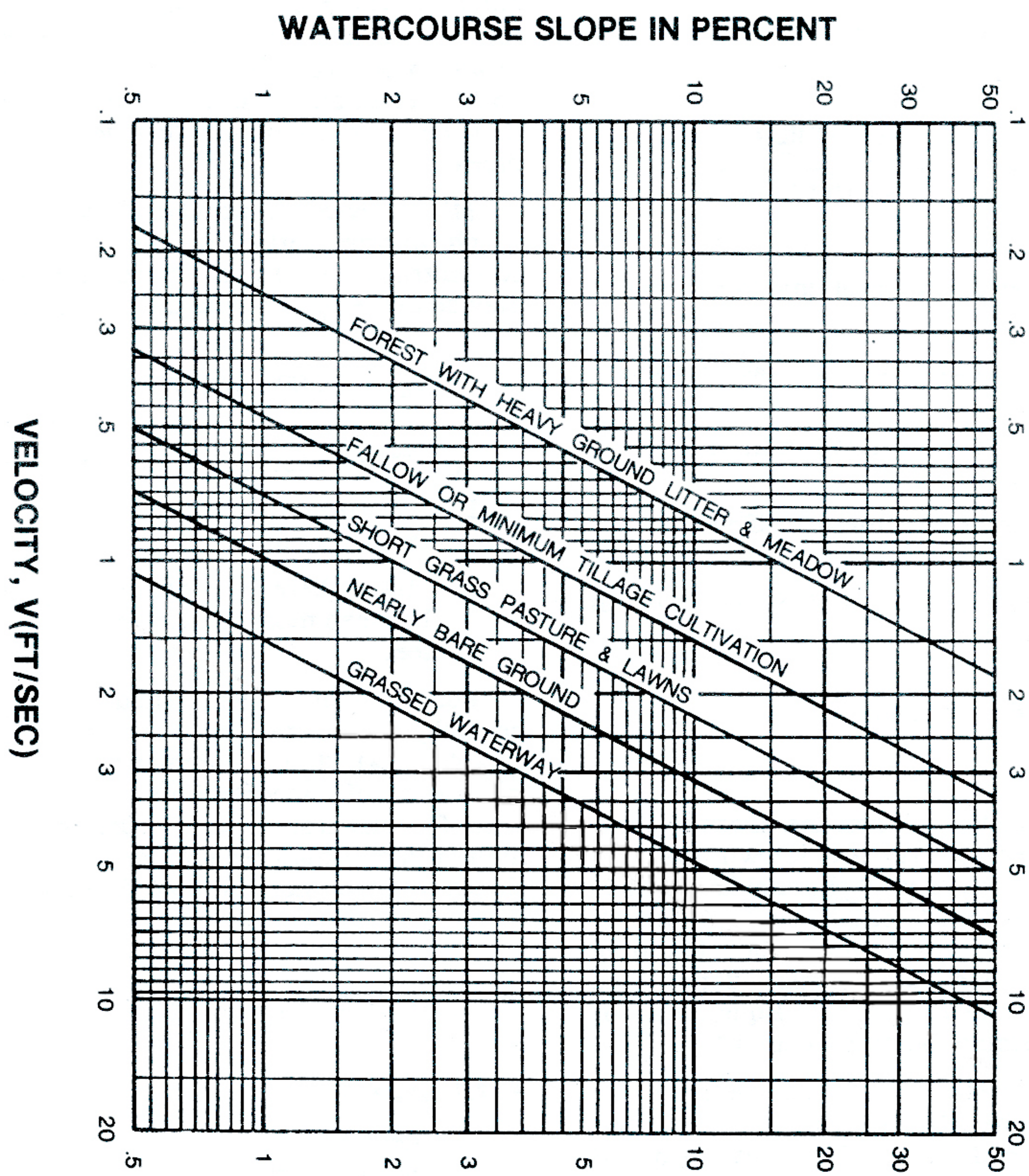


Table 816.6B
Intercept Coefficients for Shallow Concentrated Flow

Land cover/Flow regime	k
Forest with heavy ground litter; hay meadow	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland	0.152
Short grass pasture	0.213
Cultivated straight row	0.274
Nearly bare and untilled-alluvial fans	0.305
Grassed waterway	0.457

The travel time can be calculated from:

$$T_t = \frac{L}{60\,V}$$

where T_t is the travel time in minutes, L the length in feet, and V the flow velocity in feet per second.

(3) *Channel flow travel time.* When the channel characteristics and geometry are known the preferred method of estimating channel flow time is to divide the channel length by the channel velocity obtained by using the Manning equation, assuming bankfull conditions. See Index 866.3, Open Channel Flow Equations for further discussion of Manning's equation.

Appropriate values for "n", the coefficient of roughness in the Manning equation, may be found in most hydrology or hydraulics text and reference books. Table 866.3A gives some "n" values for lined and unlined channels, gutters, and medians. Procedures for selecting an appropriate hydraulic roughness coefficient may be found in the FHWA report, "Guide for Selecting Manning's Roughness Coefficient for Natural Channels and Flood Plains". Generally, the channel roughness factor will be much lower than the values for overland flow with similar surface appearance.

Culvert or Storm Drain Flow. Flow velocities in a short culvert are generally higher than they would be in the same length of natural channel and comparable to those in a lined channel. In most cases, including short runs of culvert in the channel, flow time calculation will not materially affect the overall time of concentration (T_c). When it is appropriate to separate flow time calculations, such as for urban storm drains, Manning's equation may be used to obtain flow velocities within pipes.

The TR-55 library of equations for sheet flow, shallow concentrated flow and open channel flow is incorporated into the Watershed Modeling System (WMS) for Time of Concentration Calculations using Triangulated Irregular Networks (TINs) and Digital Elevation Maps (DEMs).

Topic 817 - Flood Magnitude

817.1 General

The determination of flood magnitude from either measurements made during a flood or after peak flow has subsided requires knowledge of open-channel hydraulics and flood water behavior. There are USGS Publications and other technical references available which outline the procedures for measuring flood flow. However, it is only through experience that accurate measurements can be obtained and/or correctly interpreted.

817.2 Measurements

(1) *Direct.* Direct flood flow measurements are those made during flood stage. The area and average velocity can be approximated and the estimated discharge can be calculated, from measurements of flow depth and velocity made simultaneously at a number of points in a cross section.

Discharges calculated from continuous records of stage gaging stations are the primary basis for estimating the recurrence interval or frequency of floods.

(2) *Indirect.* Indirect flood flow measurements are those made after the flood subsides. From channel geometry measurements and high water marks the magnitude of a flood can be calculated using basic open channel hydraulic equations given in Chapter 860. This method of determining flood discharges for given events is a valuable tool to the highway engineer possessing a thorough knowledge and understanding of the techniques involved.

Topic 818 - Flood Probability And Frequency

818.1 General

The estimation of peak discharges of various recurrence intervals is the most common and important problem encountered in highway engineering hydrology. Since the hydrology for the sizing of highway drainage facilities is concerned with future events, the time and magnitude of which cannot be precisely forecast, the highway engineer must resort to probability statistics to define the design discharge.

Modern hydrologists tend to define floods in terms of probability, as expressed in percentage rather than in terms of return period (recurrence interval). Return period, the "N-year flood", and probability (p) are reciprocals, that is, $p = 1/N$. Therefore, a flood having a 50-year return frequency (Q_{50}) is now commonly expressed as a flood with the probability of recurrence of 0.02 (2 percent chance of being exceeded) in any given year.

There are certain other terminologies which are frequently used and understood by highway engineers but which might have a slight variation in meaning to other engineering branches. For convenience and example, the following definition of terms have been excerpted from Topic 806, Definition of Drainage Terms.

(1) *Base Flood.* "The flood or tide having a 1 percent chance of being exceeded in any given year". The "base flood" is commonly used as the standard flood in Federal insurance studies and has been adopted by many agencies for flood hazard analysis to comply with regulatory requirements. See Topic 804, Floodplain Encroachments.

(2) *Overtopping Flood.* "The flood described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief". The "overtopping flood" is of particular interest to highway drainage engineers because it may be the threshold where the relatively low profile of the highway acts as a flood relief mechanism for the purpose of minimizing upstream backwater damages.

(3) *Design Flood.* "The peak discharge (when appropriate, the volume, stage, or wave crest elevation) of the flood associated with the probability of exceedance selected for the design of a highway encroachment". Except for the rare situation where the risks associated with a low water crossing are acceptable, the highway will not be inundated by the "design flood".

(4) *Maximum Historical Flood.* "The maximum flood that has been recorded or experienced at any particular highway location". This information is very desirable and where available is an indication that the flood of this magnitude may be repeated at the project site. Hydrologic analysis may suggest that the probability for recurrence of the "maximum historical flood" is very small, less than 1 percent. Nevertheless consideration should be given to sizing drainage structures to convey the "maximum historical flood".

(5) *Probable Maximum Flood.* "The flood discharge that may be expected from the most severe combination of critical meteorological and hydrological conditions that are reasonably possible in the region". The "probable maximum flood" is generally not applicable to highway projects. The possibility of a flood of such rare magnitude, as used by the Corps of Engineers, is applicable to projects such as major dams, when consideration is to be given to virtually complete security from potential floods.

818.2 Establishing Design Flood Frequency

There are two recognized alternatives to establishing an appropriate highway drainage design frequency. That is, by policy or by economic analysis. Both alternatives have merit and may be applied exclusively or jointly depending upon general conditions or specific constraints.

Application of traditional predetermined design flood frequencies implies that an acceptable level of risk was considered in establishing the design standard. Modern design concepts, on the other hand, recommend that a range of peak flows be considered and that the design flood be established which best satisfies the specific site conditions and associated risks. A preliminary evaluation of the inherent flood-related risks to upstream and downstream properties, the highway facility, and to the traveling public should be made. This evaluation will indicate whether a predetermined design flood frequency is applicable or additional study is warranted.

Highway classification is one of the most important factors, but not the sole factor, in establishing an appropriate design flood frequency. Due consideration should be given to all the other factors listed under Index 801.5. If the analysis is correct, the highway drainage system will occasionally be overtaxed. The alternative of accommodating the worst possible event that could happen is usually so costly that it may not be justified.

Highway engineers should understand that the option to select a predetermined design flood frequency is generally only applicable to new highway locations. Because of existing constraints, the freedom to select a prescribed design flood frequency may not exist for projects involving replacement of existing facilities. Caltrans policy relative to up-grading of existing drainage facilities may be found in Index 803.3.

Although the procedures and methodology presented in HEC 17, Design of Encroachments on Flood Plains Using Risk Analysis, are not fully endorsed by Caltrans, the circular is an available source of information on the theory of "least total expected cost (LTEC) design". Highway engineers are cautioned about applying LTEC methodology and procedures to ordinary drainage design problems. The Headquarters Hydraulics Engineer in the Division of Design should be consulted before committing to design by the LTEC method since its use can only be justified and recommended under extra-ordinary circumstances.

Topic 819 - Estimating Design Discharge

819.1 Introduction

Before highway drainage facilities can be hydraulically designed, the quantity of run-off (design Q) that they may reasonably be expected to convey must be established. The estimation of peak discharge for various recurrence intervals is therefore the most important, and often the most difficult, task facing the highway engineer. Refer to Table 819.5A for a summary of methods for estimating design discharge.

In Topic 819, various design recommendations are given for both general and region-specific areas of California.

819.2 Empirical Methods

Because the movement of water is so complex, numerous empirical methods have been used in hydrology. Empirical methods in hydrology have great usefulness to the highway engineer. When correctly applied by engineers knowledgeable in the method being used and its idiosyncrasies, peak discharge estimates can be obtained which are functionally acceptable for the design of highway drainage structures and other features. Some of the more commonly used empirical methods for estimating runoff are as follows.

(1) *Rational Methods*. Undoubtedly, the most popular and most often misused empirical hydrology method is the Rational Formula:

$$Q = CiA$$

Q = Design discharge in cubic feet per second.

C = Coefficient of runoff.

i = Average rainfall intensity in inches per hour for the selected frequency and for a duration equal to the time of concentration. See http://hdsc.nws.noaa.gov/hdsc/pfds/

A = Drainage area in acres.

Rational methods are simple to use, and it is this simplicity that has made them so popular among highway drainage design engineers. Design discharge, as computed by these methods, has the same probability of occurrence (design frequency) as the frequency of the rainfall used. Refer to Topic 818 for further information on flood probability and frequency of recurrence.

An assumption that limits applicability is that the rainfall is of equal intensity over the entire watershed. Because of this, Rational Methods should be used only for estimating runoff from small simple watershed areas, preferably no larger than 320 acres. Even where the watershed area is relatively small but complicated by a mainstream fed by one or more significant tributaries, Rational Methods should be applied separately to each tributary stream and the tributary flows then routed down the main channel. Flow routing can best be accomplished through the use of hydrographs discussed under Index 816.5. Since Rational Methods give results that are in terms of instantaneous peak discharge and provide little information relative to runoff rate with respect to time, synthetic hydrographs should be developed for routing significant tributary inflows. Several relatively simple methods have been established for developing hydrographs, such as transposing a hydrograph from another hydrologically homogeneous watershed. The stream hydraulic method, and upland method are described in HDS No. 2. These, and other methods, are adequate for use with Rational Methods for estimating peak discharge and will provide results that are acceptable to form the basis for design of highway drainage facilities.

It is clearly evident upon examination of the assumptions and parameters which form the basis of the equation that much care and judgment must be applied with the use of Rational Methods to obtain reasonable results.

- The runoff coefficient "C" in the equation represents the percent of water which will run off the ground surface during the storm. The remaining amount of precipitation is lost to infiltration, transpiration, evaporation and depression storage.

Values of "C" may be determined for undeveloped areas from Figure 819.2A by considering the four characteristics of: relief, soil infiltration, vegetal cover, and surface storage.

Some typical values of "C" for developed areas are given in Table 819.2B. Should the basin contain varying amounts of different cover, a weighted runoff coefficient for the entire basin can be determined as:

$$C = \frac{C_1 A_2 + C_2 A_2 + ...}{A_1 + A_2 + ...}$$

- To properly satisfy the assumption that the entire drainage area contributes to the flow; the rainfall intensity, (i) in the equation expressed in inches per hour, requires that the storm duration and the time of concentration (t_c) be equal. Therefore, the first step in estimating (i) is to estimate (t_c). Methods for determining time of concentration are discussed under Index 816.6.
- Once the time of concentration, (t_c), is estimated, the rainfall intensity, (i), corresponding to a storm of equal duration, may be obtained from available sources such as intensity-duration-frequency (IDF) curves. See Index 819.6 for recommendations regarding IDF curve generating software.

The runoff coefficients given in Figure 819.2A and Table 819.2B are applicable for storms of up to 5 or 10 year frequencies. Less frequent, higher intensity storms usually require modification of the coefficient because infiltration, detention, and other losses have a proportionally smaller effect on the total runoff volume. The adjustment of the rational method for use with major storms can be made by multiplying the coefficient by a frequency factor, C(f). Values of C(f) are given below. Under no circumstances should the product of C(f) times C exceed 1.0.

Frequency (yrs)	C(f)
25	1.1
50	1.2
100	1.25

(2) *Regional Analysis Methods.* Regional analysis methods utilize records for streams or drainage areas in the vicinity of the stream under consideration which would have similar characteristics to develop peak discharge estimates. These methods provide techniques for estimating annual peak stream discharge at any site, gaged or ungaged, for probability of recurrence from 50 percent (2 years) to 1 percent (100 years). Application of these methods is convenient, but the procedure is subject to some limitations.

Regional Flood - Frequency equations developed by the U.S. Geological Survey for

Figure 819.2A

Runoff Coefficients for Undeveloped Areas

Watershed Types

	Extreme	High	Normal	Low
Relief	.28 -.35 Steep, rugged terrain with average slopes above 30%	.20 -.28 Hilly, with average slopes of 10 to 30%	.14 -.20 Rolling, with average slopes of 5 to 10%	.08 -.14 Relatively flat land, with average slopes of 0 to 5%
Soil Infiltration	.12 -.16 No effective soil cover, either rock or thin soil mantle of negligible infiltration capacity	.08 -.12 Slow to take up water, clay or shallow loam soils of low infiltration capacity, imperfectly or poorly drained	.06 -.08 Normal; well drained light or medium textured soils, sandy loams, silt and silt loams	.04 -.06 High; deep sand or other soil that takes up water readily, very light well drained soils
Vegetal Cover	.12 -.16 No effective plant cover, bare or very sparse cover	.08 -.12 Poor to fair; clean cultivation crops, or poor natural cover, less than 20% of drainage area over good cover	.06 -.08 Fair to good; about 50% of area in good grassland or woodland, not more than 50% of area in cultivated crops	.04 -.06 Good to excellent; about 90% of drainage area in good grassland, woodland or equivalent cover
Surface Storage	.10 -.12 Negligible surface depression few and shallow; drainageways steep and small, no marshes	.08 -.10 Low; well defined system of small drainageways; no ponds or marshes	.06 -.08 Normal; considerable surface depression storage; lakes and pond marshes	.04 -.06 High; surface storage, high; drainage system not sharply defined; large floodplain storage or large number of ponds or marshes
Given	An undeveloped watershed consisting of; 1) rolling terrain with average slopes of 5%, 2) clay type soils, 3) good grassland area, and 4) normal surface depressions.		Solution: Relief 0.14 Soil Infiltration 0.08 Vegetal Cover 0.04 Surface Storage <u>0.06</u> C = 0.32	
Find	The runoff coefficient, C, for the above watershed.			

Table 819.2B

Runoff Coefficients for Developed Areas

Type of Drainage Area	Runoff Coefficient
Business:	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential:	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries:	0.10 - 0.25
Playgrounds:	0.20 - 0.40
Railroad yard areas:	0.20 - 0.40
Unimproved areas:	0.10 - 0.30
Lawns:	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average, 2-7%	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2-7%	0.18 - 0.25
Heavy soil, steep, 7%	0.25 - 0.35
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85
Drives and walks	0.75 - 0.85
Roofs:	0.75 - 0.95

use in California are given in Figure 819.2C and Table 819.7A. These equations are based on regional regression analysis of data from stream gauging stations. The equations in Figure 819.2C were derived from data gathered and analyzed through the mid-1970’s, while the regions covered by Table 819.7A are reflective of a more recent (1994) study of the Southwestern U.S, which has been supplemented by a 2007 Study of California Desert Region Hydrology. Nomographs and complete information on use and development of this method may be found in "Magnitude and Frequency of Floods in California" published in June, 1977 by the U.S. Department of the Interior, Geological Survey.

The Regional Flood-Frequency equations are applicable only to sites within the flood-frequency regions for which they were derived and on streams with virtually natural flows. For example, the equations are not generally applicable to small basins on the floor of the Sacramento and San Joaquin Valleys as the annual peak data which are the basis for the regression analysis were obtained principally in the adjacent mountain and foothill areas. Likewise, the equations are not directly applicable to streams in urban areas affected substantially by urban development. In urban areas the equations may be used to estimate peak discharge values under natural conditions and then by use of the techniques described in the publication or HDS No. 2, adjust the discharge values to compensate for urbanization. Further limitations on the use of USGS Regional Flood-Frequency equations are:

Region	Drainage Area (A) mi^2	Mean Annual Precip (P) in	Altitude Index (H) 1000 ft
(1)North Coast	0.2-3000	19-104	0.2-5.7
(2)Northeast	0.2-25	all	all
Sierra	0.2-9000	7-85	0.1-9.7
Central Coast	0.2-4000	8-52	0.1-2.4
South Coast	0.2-600	7-40	all
(3)South Lahontan-Colorado Desert	N/A	N/A	N/A

Notes:

(1) In the North Coast region use a minimum value of 1 for altitude index (H)

(2) See Index 819.7 for hydrologic procedures for those portions of the Northeast Region classified as desert.

(3) USGS equations not recommended. See Index 819.7

A method for directly estimating design discharges for some gaged and ungaged streams is also provided in HDS No. 2. The method is applicable to streams on or nearby those for which study data are available.

(3) Flood Frequency Analysis

(a) If there are two gaged sites with similar watershed characteristics but one has a short record and the other has a longer record of peak flows, a two-station comparison analysis can be conducted to extend the equivalent length of record at the shorter gaged site.

(b) Flood-frequency relations at sites near gaged sites on the same stream (or in a similar watershed) can be estimated using a ratio of drainage area for the ungaged and gaged sites.

(c) At a gaged site, weighted estimates of peak discharges based on the station flood-frequency relation and the regional regression equations are considered the best estimates of flood frequency and are used to reduce the time-sampling error that may occur in a station flood-frequency estimate.

(d) The flood-frequency flows and the maximum peak discharges at several stations in a region should be used whenever possible for comparison with the peak discharge estimated at an ungaged site using a rainfall-runoff approach or regional regression equation. The watershed characteristics at the ungaged and gaged sites should be similar.

(4) National Resources Conservation Service (NRCS) Methods. The Soil Conservation Service's SCS (former title) National Engineering Handbook, 1972, and their 1975, "Urban Hydrology for Small Watersheds", Technical Release 55 (TR-55), present a graphical method for estimating peak discharge. Most NRCS equations and curves provide results in terms of inches of runoff for unit hydrograph development and are not applicable to the estimation of a peak design discharge unless the design hydrograph is first developed in accordance with prescribed NRCS procedures. NRCS methods and procedures are applicable to drainage areas less than 3 square miles (approx. 2,000 acres) and result in a design hydrograph and design discharge that are functionally acceptable to form the basis for the design of highway drainage facilities.

819.3 Statistical Methods

Statistical methods of predicting stream discharge utilize numerical data to describe the process. Statistical methods, in general, do not require as much subjective judgment to apply as the previously described deterministic methods. They are usually well documented mathematical procedures which are applied to measured or observed data. The accuracy of statistical methods can also be measured quantitatively. However, to assure that statistical method results are valid, the method and procedures used should be verified by an experienced engineer with a thorough knowledge of engineering statistics.

Analysis of gaged data permits an estimate of the peak discharge in terms of its probability or

Figure 819.2C
Regional Flood-Frequency Equations

NORTH COAST REGION[2]

$Q_2 = 3.52\ A^{0.90}\ P^{0.89}\ H^{-0.87}$
$Q_5 = 5.04\ A^{0.89}\ P^{0.91}\ H^{-0.35}$
$Q_{10} = 6.21\ A^{0.88}\ P^{0.93}\ H^{-0.27}$
$Q_{25} = 7.64\ A^{0.87}\ P^{0.94}\ H^{-0.17}$
$Q_{50} = 8.57\ A^{0.87}\ P^{0.96}\ H^{-0.08}$
$Q_{100} = 9.23\ A^{0.87}\ P^{0.97}$

NORTHEAST REGION[3,4]

$Q_2 = 22\ A^{0.40}$
$Q_5 = 46\ A^{0.45}$
$Q_{10} = 61\ A^{0.49}$
$Q_{25} = 84\ A^{0.54}$
$Q_{50} = 103\ A^{0.57}$
$Q_{100} = 125\ A^{0.59}$

SOUTH LAHONTAN-COLORADO DESERT REGION[3,4]

$Q_2 = 7.3\ A^{0.30}$
$Q_5 = 53.0\ A^{0.44}$
$Q_{10} = 150\ A^{0.53}$
$Q_{25} = 410.0\ A^{0.63}$
$Q_{50} = 700.0\ A^{0.68}$
$Q_{100} = 1080.0\ A^{0.71}$

SIERRA REGION

$Q_2 = 0.24\ A^{0.88}\ P^{1.58}\ H^{-0.80}$
$Q_5 = 1.20\ A^{0.82}\ P^{1.37}\ H^{-0.64}$
$Q_{10} = 2.63\ A^{0.80}\ P^{1.25}\ H^{-0.58}$
$Q_{25} = 6.55\ A^{0.79}\ P^{1.12}\ H^{-0.52}$
$Q_{50} = 10.4\ A^{0.78}\ P^{1.06}\ H^{-0.48}$
$Q_{100} = 15.7\ A^{0.77}\ P^{1.02}\ H^{-0.43}$

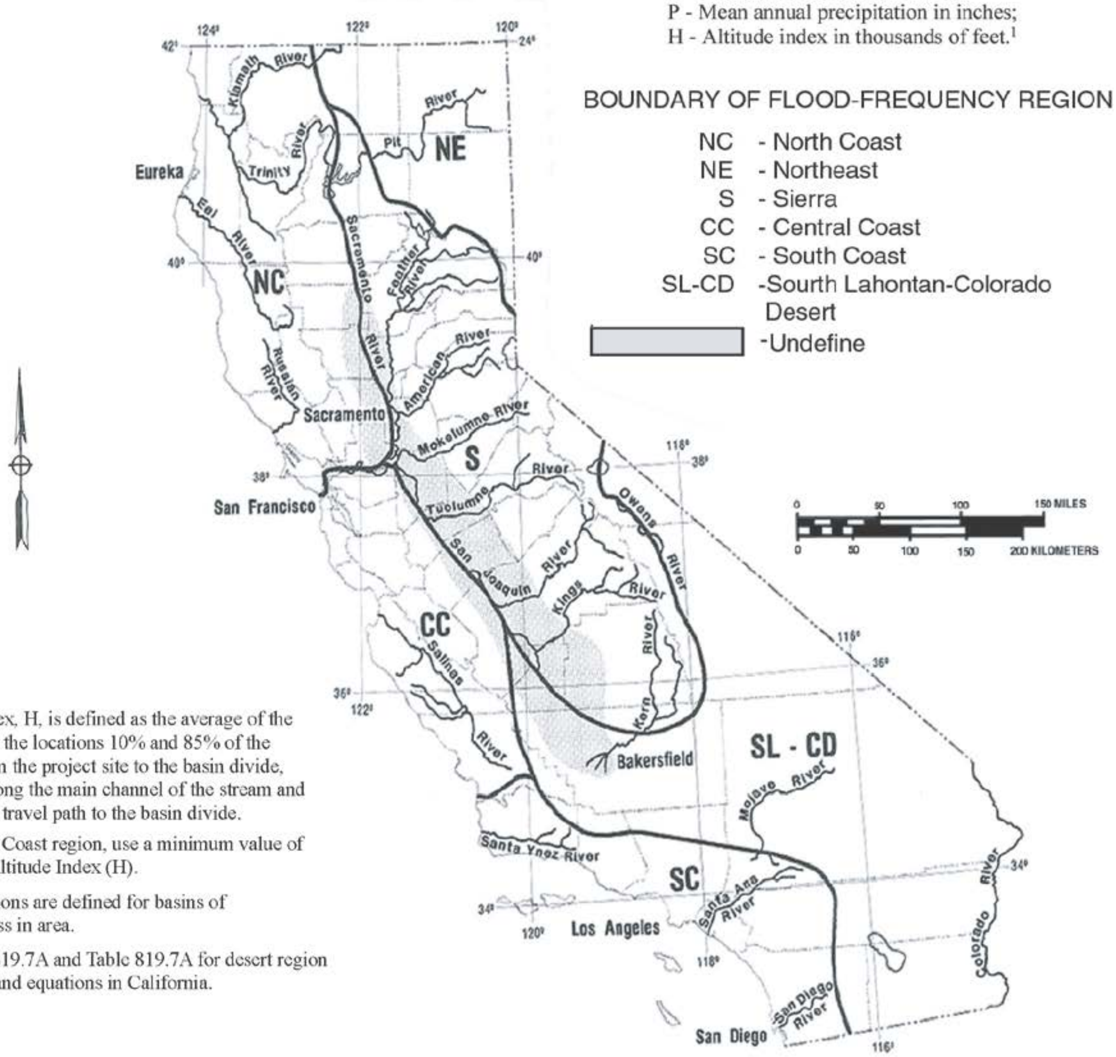
CENTRAL COAST REGION

$Q_2 = 0.0061\ A^{0.92}\ P^{2.54}\ H^{-1.10}$
$Q_5 = 0.118\ A^{0.91}\ P^{1.95}\ H^{-0.79}$
$Q_{10} = 0.583\ A^{0.90}\ P^{1.61}\ H^{-0.64}$
$Q_{25} = 2.91\ A^{0.89}\ P^{1.26}\ H^{-0.50}$
$Q_{50} = 8.20\ A^{0.89}\ P^{1.03}\ H^{-0.41}$
$Q_{100} = 19.7\ A^{0.88}\ P^{0.84}\ H^{-0.33}$

SOUTH COAST REGION

$Q_2 = 0.14\ A^{0.72}\ P^{1.62}$
$Q_5 = 0.40\ A^{0.77}\ P^{1.69}$
$Q_{10} = 0.63\ A^{0.79}\ P^{1.75}$
$Q_{25} = 1.10\ A^{0.81}\ P^{1.81}$
$Q_{50} = 1.50\ A^{0.82}\ P^{1.85}$
$Q_{100} = 1.95\ A^{0.83}\ P^{1.87}$

Q - Peak discharge in CFS, subscript indicates recurrence interval, in years;
A - Drainage area in square miles;
P - Mean annual precipitation in inches;
H - Altitude index in thousands of feet.[1]



NOTES:

1. Altitude Index, H, is defined as the average of the elevations at the locations 10% and 85% of the distance from the project site to the basin divide, measured along the main channel of the stream and the overland travel path to the basin divide.
2. In the North Coast region, use a minimum value of 1.0 for the Altitude Index (H).
3. These equations are defined for basins of 25 mi^2 or less in area.
4. See Figure 819.7A and Table 819.7A for desert region delineation and equations in California.

frequency of recurrence at a given site. This is done by statistical methods provided sufficient data are available at the site to permit a meaningful statistical analysis to be made. Water Resources Council Bulletin 17B, 1981, suggests at least 10 years of record are necessary to arrant astatistical analysis. The techniques of inferential statistics, the branch of statistics dealing with the inference of population characteristics, are described in HDS No. 2.

Before data on the specific characteristics to be examined can be properly analyzed, it must be arranged in a systematic manner. Several computer programs are available which may be used to systematically arrange data and perform the statistical computations.

Some common types of data groupings are as follows:

- Magnitude
- Time of Occurrence
- Geographic Location

Several standard frequency distributions have been studied extensively in the statistical analysis of hydrologic data. Those which have been found to be most useful are:

(1) *Log-Pearson Type III Distribution.* The popularity of the Log-Pearson III distribution is simply based on the fact that it very often fits the available data quite well, and it is flexible enough to be used with a wide variety of distributions. Because of this flexibility, the U.S. Water Resources Council recommends its use by all U.S. Government agencies as the standard distribution for flood frequency studies.

The three parameters necessary to describe the Log-Pearson III distribution are:

- Mean flow
- Standard deviation

Coefficient of skew Log-Pearson III distributions are usually plotted on log-normal probability graph paper for convenience even though the plotted frequency distribution may not be a straight line.

(2) *Log-normal Distribution.* The characteristics of the log-normal distribution are the same as those of the classical normal or Gaussian mathematical distribution except that the flood flow at a specified frequency is replaced with its logarithm and has a positive skew. Positive skew means that the distribution is skewed toward the high flows or extreme values.

(3) *Gumbel Extreme Value Distribution.* The characteristics of the Gumbel extreme value distribution (also known as the double exponential distribution of extreme values) are that the mean flood occurs at the return period of T_r = 2.33 years and that it has a positive skew.

Special probability paper has been developed for plotting log-normal and Gumbel distributions so that sample data, if it is distributed according to prescribed equations, will plot as a straight line.

819.4 Hydrograph Methods

Hydrograph methods of estimating design discharge relate runoff rates to time in response to a design storm. When storage must be considered, such as in reservoirs, natural lakes, and detention basins used for drainage or sediment control, the volume of runoff must be known. Since the hydrograph is a plot of flow rate against time, the area under the hydrograph represents volume. If streamflow and precipitation records are available for a particular design site, the development of the design hydrograph is a straight forward procedure. Rainfall records can be readily analyzed to estimate unit durations and the intensity which produces peak flows near the desired design discharge.

Hydrographs are also useful for determining the combined rates of flow for two drainage areas which peak at different times. Hydrographs can also be compounded and lagged to account for complex storms of different duration and varying intensities. Several methods of developing hydrographs are described in HDS No. 2. For basins without data, two of the most widely used methods described in HDS No. 2 for developing synthetic hydrographs are:

- Unit Hydrograph
- SCS Triangular Hydrograph

Both methods however tend to be somewhat inflexible since storm duration is determined by empirical relations.

819.5 Transfer of Data

Often the highway engineer is confronted with the problem where stream flow and rainfall data are not available for a particular site but may exist at points upstream or in an adjacent or nearby watersheds.

(a) If the site is on the same stream and near a gaging station, peak discharges at the gaging station can be adjusted to the site by drainage area ratio and application of some appropriate power to each drainage area. The USGS may be helpful in suggesting appropriate powers to be used for a specific hydrologic region.

(b) If a design hydrograph can be developed at an upstream point in the same watershed, the procedure described in HDS No. 2 can be used to route the design hydrograph to the point of interest.

(c) IDF curve generating software, such as NOAA’s Atlas 14, have internal routines that provide interstation interpolation that accounts not only for distance from gauge stations, but other factors, such as elevation. No additional effort is required by the designer to address distance/location effects.

819.6 Hydrologic Computer Programs

The rapid advancement of computer technology in recent years has resulted in the development of many mathematical models for the purpose of calculating runoff and other hydrologic phenomena. In the hands of knowledgeable and experienced engineers, good computer models are capable of efficiently calculating discharge estimates and other hydrologic results that are far more reliable than those which were obtained by other means. On the other hand, there is a tendency for the inexperienced engineer to accept computer generated output without questioning the reasonableness of the results obtained from a hydrologic viewpoint. Most computer simulation models require a significant amount of input data that must be carefully examined by a competent and experienced user to assure reliable results.

Some hydrologic computer models merely solve empirical hand methods more quickly. Other models are theoretical and solve the entire runoff cycle using mathematical equations to represent each phase of the runoff cycle.

In most simulation models, the drainage area is divided into subareas with similar hydrologic characteristics. A design rainfall is synthesized for each subarea, abstractions removed, and an overland flow routine simulates the movement of surface water into channels. The channels of the watershed are linked together and the channel flow is routed through them to complete the basin's response to the design rainstorm. Simulation models require calibration of modeling parameters using measured historical events to increase their validity.

A summary of personal computer programs is listed in Table 808.1.

Watershed Modeling System (WMS) is a comprehensive environment for hydrologic analysis. It was developed by the Engineering Computer Graphics Laboratory of Brigham Young University in cooperation with the U.S. Army Corps of Engineers Waterways Experiment Station (WES).

WMS merges information obtained from terrain models and GIS with industry standard hydrologic analysis models such as HEC-1 and TR-55. HY-8 has also been incorporated for culvert design.

Terrain models can obtain geometric attributes such as area, slope and runoff distances. Many display options are provided to aid in modeling and understanding the drainage characteristics of terrain surfaces.

The distinguishing difference between WMS and other applications designed for setting up hydrologic models like HEC-1 and TR-55 is its unique ability to take advantage of digital terrain for hydrologic data development.

WMS uses three primary data sources for model development:

1. Geographic Information Systems (GIS) Data
2. Digital Elevation Models (DEMs) published by the U.S. Geological Survey (USGS) at both 1:24,000 and 1:250,000 for the entire U.S. (the 1:24,000 data coverage is not complete)
3. Triangulated Irregular Networks (TINs)

Table 819.5A
Summary of Methods for Estimating Design Discharge

METHOD	ASSUMPTIONS	DATA NEEDS
Rational	• Small catchment (< 320 acres) • Concentration time < 1 hour • Storm duration >or = concentration time • Rainfall uniformly distributed in time and space • Runoff is primarily overland flow • Negligible channel storage	Time of Concentration Drainage area Runoff coefficient Rainfall intensity (http://hdsc.nws.noaa.gov/hdsc/pfds/)
USGS Regional Regression Equations: USGS Water-Resources Investigation 77-21* Improved Highway Design Methods for Desert Storms	• Catchment area limit varies by region • Basin not located on floor of Sacramento or San Joaquin Valleys • Peak discharge value for flow under natural conditions unaffected by urban development and little or no regulation by lakes or reservoirs • Ungaged channel	Drainage area Mean annual precipitation Altitude index
NRCS (TR55)	• Small or midsize catchment (< 3 square miles) • Concentration time range from 0.1-10 hour (tabular hydrograph method limit < 2 hour) • Runoff is overland and channel flow • Simplified channel routing • Negligible channel storage	24-hour rainfall Rainfall distribution Runoff curve number Concentration time Drainage area
Unit Hydrograph (Gaged data) Synthetic Unit Hydrograph SCS Unit Hydrograph S-Graph Unit Hydrograph	• Midsize or large catchment (0.20 square miles to 1,000 square miles) • Uniformity of rainfall intensity and duration • Rainfall-runoff relationship is linear • Duration of direct runoff constant for all uniform-intensity storms of same duration, regardless of differences in the total volume of the direct runoff. • Time distribution of direct runoff from a given storm duration is independent of concurrent runoff from preceding storms • Channel-routing techniques used to connect streamflows	Rainfall hyetograph and direct runoff hydrograph for one or more storm events Drainage area and lengths along main channel to point on watershed divide and opposite watershed centroid (Synthetic Unit Hydrograph)
Statistical (gage data) Log-Pearson Type III Bulletin #17B – U.S. Department of the Interior	• Midsized and large catchments with stream gage data • Appropriate station and/or generalized skew coefficient relationship applied • Channel storage	10 or more years of gaged flood records
Basin Transfer of Gage Data	• Similar hydrologic characteristics • Channel storage	Discharge and area for gaged watershed Area for ungaged watershed

* Magnitude and Frequency of Floods in California

Two other hydrologic computer programs that are commonly used are the Army Corps of Engineers' HEC-HMS and the National Resources Conservation Service's TR-20 Method.

Another program is the NOAA Atlas 14, a web-based IDF product. The NOAA Atlas 14 product is the preferred IDF tool for State highway projects.

819.7 Region-Specific Analysis

(1) Desert Hydrology

Figure 819.7A shows the different desert regions in California, each with distinct hydrological characteristics that will be explained in this section.

(a) Storm Type

Summer Convective Storms - In the southern desert regions (Owens Valley/Mono Lake, Mojave Desert, Sonoran Desert and the Colorado Desert), the dominant storm type is the local thunderstorm, specifically summer convective storms. These storms are characterized by their short duration, over a relatively small area (generally less than 20 mi^2), and intense rainfall, which may result in flash floods. These summer convective storms may occur at any time during the year, but are most common and intense during the summer. General summer storms can also occur over these desert regions, but are rare, and usually occur from mid-August to early October. The rainfall intensity can vary from heavy rainfall to heavy thunderstorms.

General Winter Storm - In the Antelope Valley and Northern Basin and Range regions, the dominant storm type is the general winter storm. These storms are characterized by their long duration, 6 hours to 12 hours or more, and possibly intermittently for 3 days to 5 days over a relatively large area. General winter storms produce the majority of large peaks in the northern desert areas; the majority of the largest peaks discharge greater than or equal to 20 cfs/mi^2 occurred during the winter and fall months in the Owens Valley/Mono Lake and Northern Basin and Range regions. At elevations above 6,000 ft, much of the winter precipitation falls as snow; however, snowfall doesn't play a significant role in flood-producing runoff in the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert). In the northern desert regions (Owens Valley/Mono Lake and Northern Basin and Range), more floods from snowmelt occur at lower elevations; more than 50 percent of runoff events occurred in spring, most likely snowmelt, but did not produce large floods.

(b) Regional Regression

Newly developed equations for California's Desert regions are shown on Table 819.7A.

While the regression equations for the Northern Basin and Range region provide more accurate results than previous USGS developed equations, there is some uncertainty associated with them. Therefore, the development of a rainfall-runoff model may be preferable for ungaged watersheds in this region.

(c) Rational Method

The recommended upper limit for California's desert regions is 160 acres (0.25 mi^2).

Table 819.7B lists common runoff coefficients for Desert Areas. These coefficients are applicable for storms with 2-year to 10-year return intervals, and must be adjusted for larger, less frequent storms by multiplying the coefficient by an appropriate frequency factor, C(f), as stated in Index 819.2(1) of this manual. The frequency factors, C(f), for 25-year, 50-year and 100-year storms are 1.1, 1.2 and 1.25, respectively. Under no circumstances should the product of C(f) times the runoff coefficient exceed 1.0. If a value of 1.0 is reached, it is recommended to use the value of 0.95.

(d) Rainfall-Runoff Simulation

A rainfall-runoff simulation approach uses a numerical model to simulate the rainfall-runoff process and generate discharge hydrographs. It has four main components:

rainfall; rainfall losses; transformation of effective rainfall; and channel routing.

(1) Rainfall

a. Design Rainfall Criteria

The selection of an appropriate storm duration depends on a number of factors, including the size of the watershed, the type of rainfall-runoff approach and hydrologic characteristics of the study watershed. Watershed sizes are analyzed below and are applied to California's Desert regions in Table 819.7C.

Drainage Areas $\leq$ 20 mi^2 – Drainage areas less than 20 mi^2 are primarily representative of summer convective storms, and usually occur in the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert regions). Since these storms usually result in intense rainfall, over a small drainage area and are generally less than 6 hours, it is recommended that a 6-hour local design storm be utilized.

Drainage Areas > 20 mi^2 & $\leq$ 100 mi^2 – For drainage areas between 20 mi^2 and 100 mi^2, the critical storm can be a summer convective storm or a general thunderstorm. For these drainage areas, it is recommended that both 6-hour and 24-hour design storm be analyzed, and the storm that produces the largest peak discharge be chosen as the design basis.

Drainage Areas > 100 mi^2 – Since general storms usually cover a larger area and have a longer duration, for drainage areas greater than 100 mi^2, a 24-hour design storm is recommended.

b. Depth-Duration-Frequency Characteristics

In 2011, NOAA published updated precipitation-frequency estimates for all of California including the desert regions, often cited as NOAA Atlas 14. This information is available online, via the Precipitation Frequency Data Server at http://hdsc.nws.noaa.gov/hdsc/pfds/ NOAA Atlas 14 supersedes NOAA's previous effort, NOAA Atlas 2, the 2004 Atlas 14 which covered the Southwestern U.S., and California's Department of Water Resources (DWR) Bulletin No. 195, where their coverages overlap.

NOAA Atlas 14 provides a vast amount of information, which includes:

- Point Estimates
- ESRI shapefiles and ArcInfo ASCII grids
- Color cartographic maps: all possible combination of frequencies (2-year to 1,000-year) and durations (5-miunte to 60-day)
- Associated Federal Geographic Data Committee-compliant metadata
- Data series used in the analysis: annual maximum series and partial duration series
- Temporal distributions of heavy precipitation (6-hour, 12-hour, 24-hour and 96-hour)
- Seasonal exceedance graphs: counts of events that exceed the 1 in 2, 5, 10, 25, 50 and 100 annual exceedance probabilities for the 60-minute, 24-hour, 48-hour and 10-day durations

Table 819.7A

Regional Regression Equations for California's Desert Regions (Con't)

Northern Basin & Range	$Q_2 = 5.320A^{0.415}\left[\frac{H}{1000}\right]^{0.928}$ $Q_5 = 29.71A^{0.360}\left[\frac{H}{1000}\right]^{0.296}$ $Q_{10} = 85.76A^{0.314}\left[\frac{H}{1000}\right]^{-0.109}$ $Q_{25} = 275.5A^{0.253}\left[\frac{H}{1000}\right]^{-0.555}$ $Q_{50} = 616.9A^{0.281}\left[\frac{H}{1000}\right]^{-0.867}$ $Q_{100} = 1293A^{0.166}\left[\frac{H}{1000}\right]^{-1.154}$

Table 819.7B

Runoff Coefficients for Desert Areas

Type of Drainage Area	Runoff Coefficient
Undisturbed Natural Desert or Desert Landscaping (without impervious weed barrier)	0.30 – 0.40
Desert Landscaping (with impervious weed barrier)	0.55 – 0.85
Desert Hillslopes	0.40 – 0.55
Mountain Terrain (slopes greater than 10%)	0.60 – 0.80

Table 819.7C

Watershed Size for California Desert Regions

Desert Region	Duration (based on Watershed size)
Southern Regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert)	6-hour local storm (≤ 20 mi^2) 6-hour local storm and 24-hour general storm (between 20 mi^2 & 100 mi^2); use the larger peak discharge 24-hour general storm ($>$ 100 mi^2)
Northern Regions (Owens Valley/Mono Lake and Northern Basin and Range)	24-hour general storm

c. Depth-Area Reduction

Depth-area reduction is the method of applying point rainfall data from one or several gaged stations within a watershed to that entire watershed. NOAA Atlas 14 provides high resolution depth-duration frequency point data which can then be computed with other depth-duration frequency data in that cell to obtain an average depth-duration frequency over a watershed. However, as this data is available as point data, the average calculated depth-duration frequency may not represent an entire watershed. To convert this point data into watershed area, a conversion factor may be applied, of which, two methods are available: applying a reduction factor; or applying depth-area reduction curves.

NOAA is currently working on updating the reduction factors, thus, until then, the depth-area reduction curves are recommended. Two depth-area reduction curves are available: (1) the depth curves in National Weather Service's HYDRO-40 (http://www.nws.noaa.gov/oh/hdsc/PF_related_studies/TechnicalMemorandum_HYDRO40.pdf); and (2) the depth curves in NOAA Atlas 2. The general consensus is that the depth curves from HDRO-40 better represent the desert areas of California, and are recommended for the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and the Mojave Desert). For the upper regions (Owens Valley/Mono Lake and Northern Basin and Range), the curves from NOAA Atlas 2 are recommended.

The variables needed to apply depth area reduction curves to a watershed are a storm frequency (i.e., a 100-year storm), storm duration (i.e., a 30-minutes storm), and the area of a watershed. For example, if a 100-year storm with a duration of 60-minutes were to be analyzed over a desert watershed of 25 mi^2, then using Figure 819.7B, the Depth-Area Ratio would be 0.64. This ratio would then be multiplied by the averaged point-rainfall data, which would then result in the rainfall over the entire watershed.

Point rainfall data is available from NOAA Atlas 14, which must then be converted to area rainfall data. Conversions are available in two forms: (1) the National Weather Service's HYDRO-40, and (2) NOAA Atlas 2. The National Weather Service's HYDRO-40 is recommended for the southern desert regions (Colorado Desert, Sonoran Desert, Antelope Valley and Mojave Desert.) NOAA Atlas 2 is recommended for the northern desert regions (Owens Valley/Mono Lake and Northern Basin and Range).

(2) Rainfall Losses

Antecedent Moisture Condition – The Antecedent Moisture Condition (AMC) is the amount of moisture present in the soil before a rainfall event, or conversely, the amount of moisture the soil can absorb before becoming saturated (Note: the AMC is also referred to as the Antecedent Runoff Condition [ARC]). Once the soil is saturated, runoff will occur. Generally, the AMC is classified into three levels:

- AMC I – Lowest runoff potential. The watershed soils are dry enough to allow satisfactory grading or cultivation to take place.
- AMC II – Moderate runoff potential. AMC II represents an average study condition.
- AMC III – Highest runoff potential. The watershed is practically saturated from antecedent rainfall.

Because of the different storm types present in California's desert regions, AMC I is recommended as design criteria for local thunderstorms, and AMC II is recommended as design criteria for general storms.

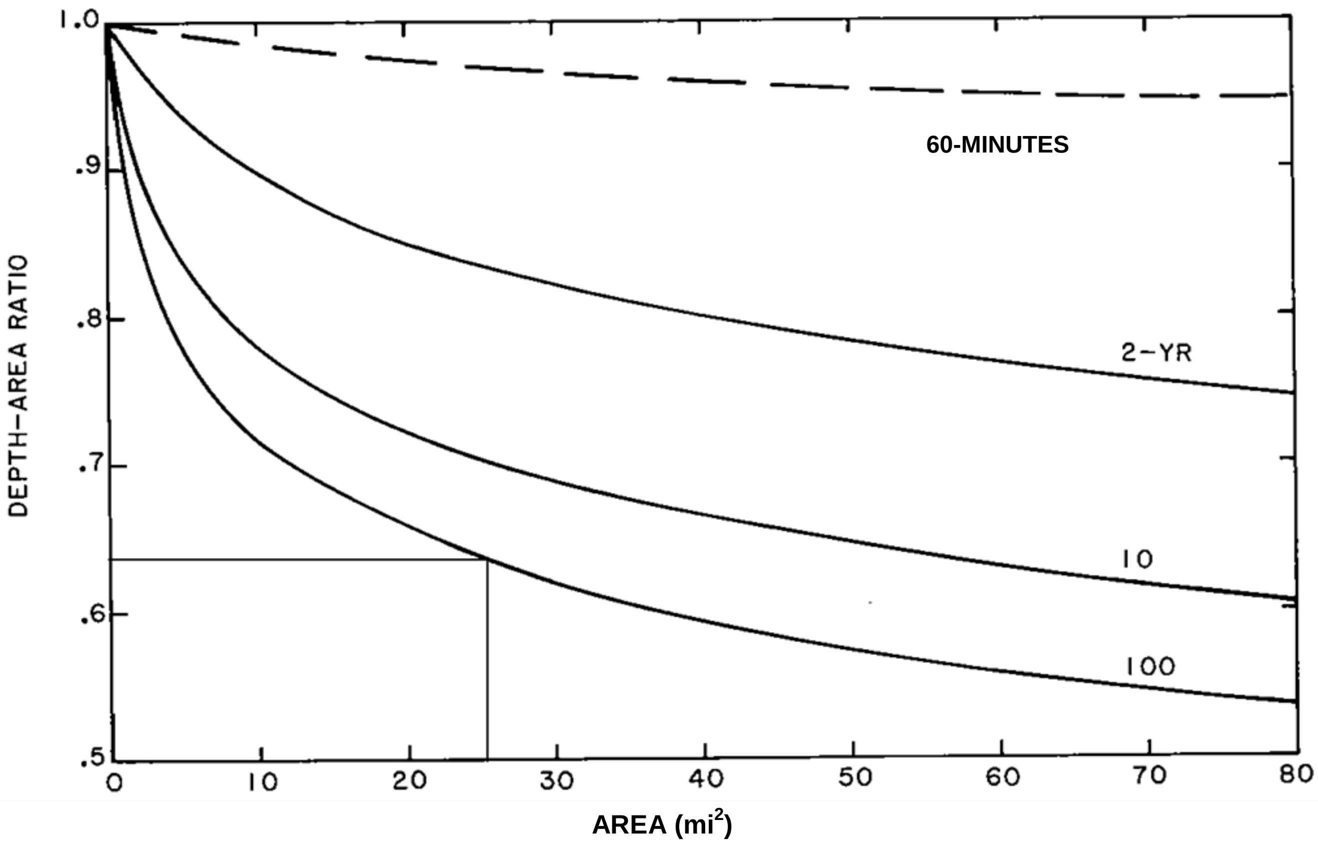
Curve Number – The curve number was developed by the then Soil Conservation Service (SCS), which is now called the National Resource Conservation Service (NRCS). The curve number is a function of land use, soil type and the soil's AMC, and is used to describe a drainage area's storm water runoff potential. The soil type(s) are typically listed by name and can be obtained in the form of a soil survey from the local NRCS office. The soil surveys classify and present the soil types into 4 different hydrological groups, which are shown in Table 819.7D. From the hydrological groups, curve numbers are assigned for each possible land use-soil group combinations, as shown in Table 819.7E. The curve numbers shown in Table 819.7E are representative of AMC II, and need to be converted to represent AMC I, and AMC III, respectively. The following equations to convert an AMC II curve number to an AMC I or AMC III curve number, using a five-day period as the minimum for estimating the AMC's:

$$CN_{AMCI} = \frac{4.2CN_{AMCII}}{10 - 0.058CN_{AMCII}}$$

$$CN_{AMCIII} = \frac{23CN_{AMCII}}{10 + 0.13CN_{AMCII}}$$

Figure 819.7B

Example Depth-Area Reduction Curve



Note: The AMC of a storm area may vary during a storm; heavy rain falling on AMC I soil can change the AMC from I to II or III during the storm.

Table 819.7D

Hydrologic Soil Groups

Hydrologic Soil Group	Soil Group Characteristics
A	Soils having high infiltration rates, even when thoroughly wetted and consisting chiefly of deep, well to excessively-drained sands or gravels. These soils have a high rate of water transmission.
B	Soils having moderate infiltration rates when thoroughly wetted and consisting of moderately deep to deep, moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C	Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.
D	Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

(3) Transformation

Total runoff can be characterized by two types of runoff flow: direct runoff and base flow. Direct runoff is classified as storm runoff occurring during or shortly after a storm event. Base flow is classified as subsurface runoff from prior precipitation events and delayed subsurface runoff from the current storm. The transformation of precipitation runoff to excess can be accomplished using a unit hydrograph approach. The unit hydrograph method is based on the assumption that a watershed, in converting precipitation excess to runoff, acts as a linear, time-invariant system.

Unit Hydrograph Approach

A unit hydrograph for a drainage area is a curve showing the time distribution of runoff that would result at the concentration point from one inch of effective rainfall over the drainage area above that point.

The unit hydrograph method assumes that watershed discharge is related to the total volume of runoff, that the time factors that affect the unit hydrograph shape are invariant, and that watershed rainfall-runoff relationships are characterized by watershed area, slope and shape factors.

a. SCS Unit Hydrogoraph

The SCS dimensionless unit hydrograph is based on averages of unit hydrographs derived from gaged rainfall and runoff for a large number of small rural basins throughout the U.S. The definition of the SCS unit hydrograph normally only requires one parameter, which is lag, defined as the time from the centroid of precipitation excess to the time of the peak of the unit hydrograph. For ungaged watersheds, the SCS suggests that the unit hydrograph lag time, t_{lag}, may be related to time

Table 819.7E

Curve Numbers for Land Use-Soil Combinations

Description	Average % Impervious	Curve Number by Hydrological Soil Group				Typical Land Uses
		A	B	C	D	
Residential (High Density)	65	77	85	90	92	Multi-Family, Apartments, Condos, Trailer Parks
Residential (Medium Density)	30	57	72	81	86	Single-Family, Lot Size ¼ to 1 acre
Residential (Low Density)	15	48	66	78	83	Single-Family, Lot Size 1 acre or greater
Commercial	85	89	92	94	95	Strip Commercial, Shopping Centers, Convenience Stores
Industrial	72	81	88	91	93	Light Industrial, Schools, Prisons, Treatment Plants
Disturbed / Transitional	5	76	85	89	91	Gravel Parking, Quarries, Land Under Development
Agricultural	5	67	77	83	87	Cultivated Land, Row Crops, Broadcast Legumes
Open Land – Good	5	39	61	74	80	Parks, Golf Courses, Greenways, Grazed Pasture
Meadow	5	30	58	71	78	Hay Fields, Tall Grass, Ungrazed Pasture
Woods (Thick Cover)	5	30	55	70	77	Forest Litter and Brush adequately cover soil
Woods (Thin Cover)	5	43	65	76	82	Light Woods, Woods-Grass Combination, Tree Farms
Impervious	95	98	98	98	98	Paved Parking, Shopping Malls, Major Roadways
Water	100	100	100	100	100	Water Bodies, Lakes, Ponds, Wetlands

of concentration t_c, through the following relation:

$$t_{lag} = 0.6t_c$$

The time of concentration is the sum of travel time through sheet flow, shallow concentrated flow, and channel flow segments. A typical SCS Unit Hydrograph is similar to Figure 816.5.

A unit hydrograph can be derived from observed rainfall and runoff, however either may be unavailable. In such cases, a synthetic unit hydrograph can be developed using the S-graph method.

a. S-graph

An S-graph is a summation hydrograph of runoff that would result from the continuous generation of unit storm effective rainfall over the area (1-inch per hour continuously). The S-graph method uses a basic time-runoff relationship for a watershed type in a form suitable for application to ungaged basins, and is based upon percent of ultimate discharge and percent of lag time. Several entities, including local and Federal agencies, have developed location-specific S-Graphs that areapplicable to California's desert regions.

The ordinate is expressed in percent of ultimate discharge, and the abscissa is expressed in percent of lag time. Ultimate discharge, which is the maximum discharge attainable for a given intensity, occurs when the rate of runoff on the summation hydrograph reaches the rate of effective rainfall.

Lag for a watershed is an empirical expression of the hydrologic characteristics of a watershed in terms of time. It is defined as the elapsed time (in hours) from the beginning of unit effective rainfall to the instant that the summation hydrograph for the point of concentration reaches 50 percent of ultimate discharge. When the lags determined from summation hydrographs for several gaged watersheds are correlated to the hydrologic characteristics of the watersheds, an empirical relationship is usually apparent. This relationship can then be used to determine the lags for comparable ungaged drainage areas for which the hydrologic characteristics can be determined, and a unit hydrograph applicable to the ungaged watersheds can be easily derived.

Figure 819.7C is a sample illustration of a San Bernardino County S-Graph, while Figure 819.7D shows an example S-Graph from USBR.

Recommendations

For watersheds with mountainous terrain/high elevations in the upper portions, the San Bernardino County Mountain S-Graph (http://www.sbcounty.gov/dpw/floodcontrol/pdf/HydrologyManual.pdf) is recommended. For watersheds in the southern desert regions with limited or no mountainous terrain/high elevations, the San Bernardino County Desert S-Graph (http://www.sbcounty.gov/dpw/floodcontrol/pdf/HydrologyManual.pdf) is recommended. The U.S. Bureau of Reclamation (USBR) S-Graph (http://www.usbr.gov/pmts/hydraulics_lab/pubs/manuals/SmallDams.pdf) is recommended for watersheds in the Northern Basin and Range.

As an alternative to the above mentioned S-Graphs, the SCS Unit Hydrograph may also be used.

Figure 819.7C

San Bernardino County Hydrograph for Desert Areas

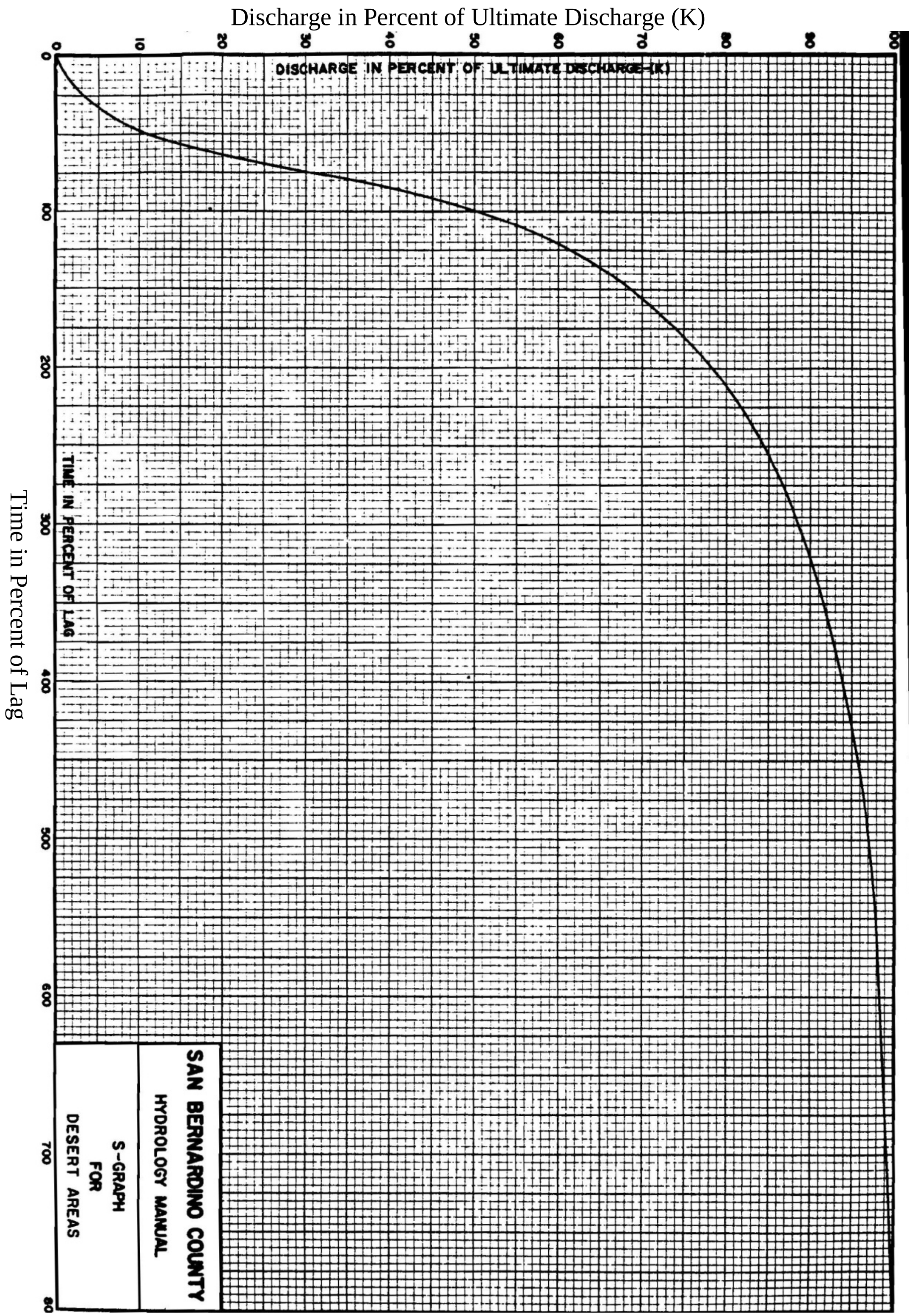


Figure 819.7D

USBR Example S-Graph

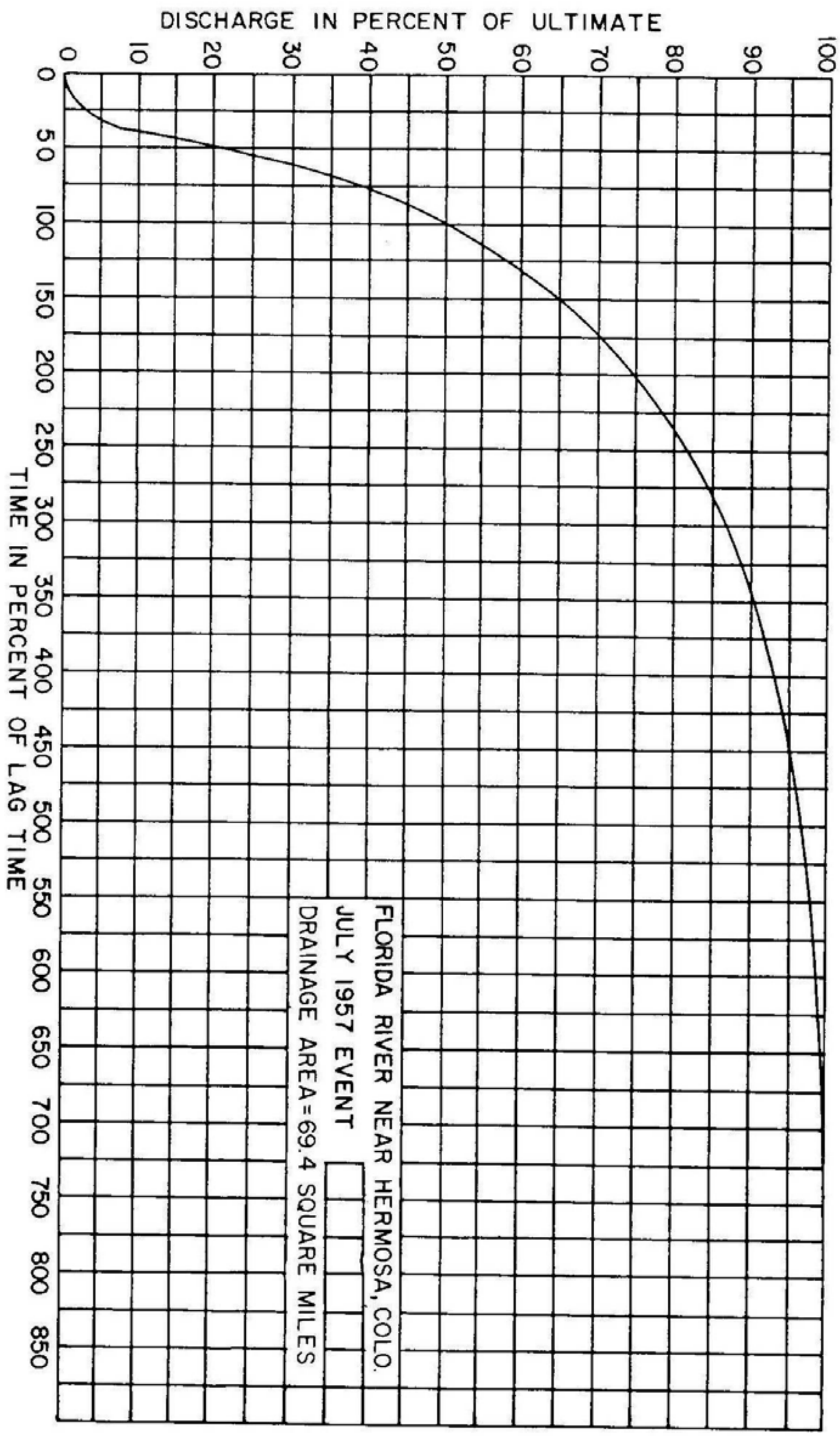


Table 819.7F

Channel Routing Methods

Routing Method	Pros	Cons
Kinetmatic Wave	▪ A conceptual model assuming a uniform flow condition. ▪ In general, works best for steep (10 ft/mile or greater), well defined channels. ▪ It is often applied in urban areas because the routing reaches are generally short and well-defined.	▪ Cannot handle hydrograph attenuation, significant overbank storage, and backwater effects.
Modified Puls	▪ Known as storage routing or level-pool routing. ▪ Can handle backwater effects through the storage-discharge relationship.	▪ Need to use hydraulic model to define the required storage-outflow relationship.
Muskingum	▪ Directly accommodates the looped relationship between storage and outflow. ▪ A linear routing technique that uses coefficients to account for hydrograph timing and diffusion.	▪ The coefficients cannot be used to model a range of floods that may remain in bank or go out of bank. Therefore, not applicable to significant overbank flows.
Muskingum-Cunge	▪ A nonlinear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. ▪ The parameters are physically based. ▪ Has been shown to compare well against the full unsteady flow equations over a wide range of flow conditions.	▪ It cannot account for backwater effects. ▪ Not very applicable for routing a very rapidly rising hydrograph through a flat channel.

(4) Channel Routing

Channel routing is a process used to predict the temporal and spatial variation of a flood hydrograph as it moves through a river reach. The effects of storage and flow resistance within a river reach are reflected by changes in hydrograph shape and timing as the flood wave moves from upstream to downstream. The four commonly used methods are the kinematic wave routing, Modified Puls routing, Muskingum routing, and Muskingum-Cunge routing. The advantages and disadvantages for each method are described in Table 819.7F. Table 819.7G provides guidance for selecting an appropriate routing method. The Muskingum-Cunge routing method can handle a wide range of flow conditions with the exception of significant backwater. The Modified Puls routing can model backwater effects. The kinematic wave routing method is often applied in urban areas with well defined channels.

(5) Storm Duration and Temporal Distribution

Temporal distribution is the time-related distribution of the precipitation depth within the duration of the design storm. Temporal distribution patterns of design storms are based on the storm duration. The temporal distribution pattern for short-duration storms represents a single cloudburst and is based on rainfall statistics. The temporal distribution for long-duration storms resembles multiple events and is patterned after historic events. Since the storm events in California's desert regions are made up of two distinct separate storm types, the summer convective storm and the general winter storm, the design storm durations should be adjusted accordingly. For California's desert regions, the 100-year 6-hour storm is recommended for the convective storms, and the 100-year 24-hour storm is recommended for the winter storms. Table 819.7H summarizes the design storm durations for the different desert regions throughout California.

Table 819.7G

Channel Method Routing Guidance

If this is true…	… then this routing model may be considered.
No observed hydrograph data available for calibration	Kinematic wave; Muskingum-Cunge
Significant backwater will influence discharge hydrograph	Modified Puls
Flood wave will go out of bank, into floodplain.	Modified Puls; Muskingum-Cunge with 8-point cross section
Channel slope > 0.002 and $\frac{TS_o u_o}{d_o} \geq 171$	Any
Channel slopes from 0.002 to 0.0004 and $\frac{TS_o u_o}{d_o} \geq 171$	Muskingum-Cunge; Modified Puls; Muskingum
Channel slope < 0.0004 and $TS_o\left(\frac{g}{d_o}\right)^{1/2} \geq 30$	Muskingum-Cunge
Channel slope < 0.0004 and $TS_o\left(\frac{g}{d_o}\right)^{1/2} < 30$	None

Notes:
T = hydrograph duration
u_o = reference mean velocity
d_o = reference flow depth
S_o = channel slope

(2) Sediment/Debris Bulking

The process of increasing the water volume flow rate to account for high concentrations of sediment and debris is defined as bulking. Debris carried in the flow can be significant and greatly increase flow volume conveyed from a watershed. This condition occurs frequently in mountainous areas subject to wildfires with soil erosion, as well as arid regions around alluvial fans and other geologic activity. By bulking the flow through the use of an appropriate bulking factor, bridge openings and culverts can be properly sized for areas that experience high sediment and debris concentration.

(a) Bulking Factor

Bulking factors are applied to a peak (clear-water) flow to obtain a total or bulked peak flow, which provides a safety factor in the sizing of hydraulic structures. For a given watershed, a bulking factor is typically a function of the historical concentration of sediment in the flow.

(b) Types of Sediment/Water Flow

The behavior of flood flows will vary depending on the concentration of sediment in the mixed flow, where the common flow types are normal stream flow, hyperconcentrated flow, and debris flow.

1. Normal Stream Flow

During normal stream flow, the sediment load minimally influences flow behavior or characteristics. Because sediment has little impact, this type of flow can be analyzed as a Newtonian fluid and standard hydraulic methods can be used. The upper limit of sediment concentration by volume for normal stream flow is 20 percent and bulking factors are applied cautiously because of the low concentration. (See Table 819.7I) The small amount of sediment is conveyed by conventional suspended load and bed-load.

2. Hyperconcentrated Flow

Hyperconcentrated flow is more commonly known as mud flow. Because of potential for large volumes of sand in the water column, fluid properties and transport characteristics change and the mixture does not behave as a Newtonian fluid. However, basic hydraulic methods and models are still generally accepted and used for up to 40 percent sediment concentration by volume. For hyperconcentrated flow, bulking factors vary between 1.43 and 1.67 as shown in Table 819.7I.

3. Debris Flow

In debris flow state, behavior is primarily controlled by the composition of the sediment and debris mixture, where the volume of clay can have a strong influence in the yield strength of the mixture.

During debris flow, which has an upper limit of 50 percent sediment concentration by volume, the sediment/debris/water mixture no longer acts as a Newtonian fluid and basic hydraulic equations do not apply. If detailed hydraulic analysis or modeling of a stream operating under debris flow is needed, FLO2DH is the recommended software choice given its specific debris flow capabilities. HEC-RAS is appropriate for normal stream flow and hyperconcentrated flow, but cannot be applied to debris flow.

For a typical debris flow event, clear-water flow occurs first, followed by a frontal wave of mud and debris. Low frequency events, such as the 100-year flood, most likely contain too much water to produce a debris flow event. Normally, smaller higher frequency events such as 10-year or 25-year floods actually have a greater probability of yielding a debris flow event requiring a higher bulking factor.

As outlined in Table 819.7I, bulking factors for debris flow vary between 1.67 and 2.00.

(c) Sediment/Debris Flow Potential

1. Debris Hazard Areas

Mass movement of rock, debris, and soil is the main source of bulked flows. This can occur in the form of falls, slides, or flows. The volume of sediment and debris from mass movement can enter streams depending upon hydrologic and geologic conditions.

The location of these debris-flow hazards include:

(1) At or near the toe of slope 2:1 or steeper

(2) At or near the intersection of ravines and canyons

(3) Near or within alluvial fans

(4) Soil Slips

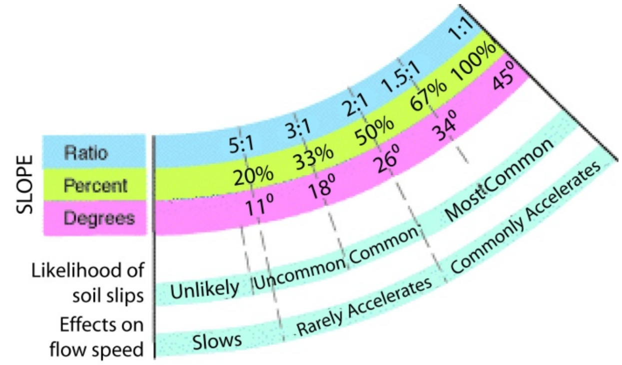
Soil slips commonly occur at toes of slope between 2:1 and 3:1. Flowing mud and rocks will accelerate down a slope until the flow path flattens. Once energy loss occurs, rock, mud, and vegetation will be deposited. Debris flow triggered by soil slips can become channelized and travel distances of a mile or more. Figure 819.7E shows the potential of soil slip versus slope angle. As seen in this Figure, the flatter the slope angle, the less effect on flow speed and acceleration.

2. Geologic Conditions

In the Transverse Ranges that include the San Gabriel and San Bernardino Mountains along the southern and southwestern borders of the Antelope Valley (Region 3) and Mojave Desert (Region 4), their substrate contains sedimentary rocks, fractured basement rocks, and granitic rocks. This type of geology has a high potential of debris flow from the hillsides of these regions.

Figure 819.7E

Soil Slips vs. Slope Angle



While debris flow potential is less prevalent, it is possible to have this condition in the Peninsula Ranges that include the San Jacinto, Santa Rosa, and Laguna Mountains along the western border of the Colorado Desert (Region 1).

(d) Alluvial Fans

An alluvial fan is a landform located at the mouth of a canyon, formed in the shape of a fan, and created over time by deposition of alluvium. With the apex of the fan at the mouth of a canyon, the base of the fan is spread across lower lying plains below the apex. Over time, alluvial fans change and evolve when sediment conveyed by flood flows or debris flows is deposited in active channels, which creates a new channel within the fan. Potentially, alluvial fan flood and debris flows travel at high velocity, where large volumes of sediment can be eroded from mountain canyons down to the lower fan surface. Given this situation, the alignments of the active channels and the overall footprint of an alluvial fan are dynamic. Also, the concentration of sediment/debris volume is dynamic, ranging from negligible to 50 percent.

Alluvial fans can be found on soil maps, geologic maps, topographic maps, and aerial photographs, in addition to the best

Table 819.7H

Design Storm Durations

Drainage Area	Desert Region	100-year, 6-hour Convective Storm (AMC I)	100-year, 24-hour General Storm (AMC II)	Regional Regression Equations
> 20 mi^2	Colorado Desert	X		
	Sonoran Desert	X		
	Mojave Desert	X		
	Antelope Valley Desert	X		
< 20 mi^2	Colorado Desert	X*	X*	
	Sonoran Desert	X*	X*	
	Mojave Desert	X*	X*	
	Antelope Valley Desert	X*	X*	
	Owens Valley/Mono Lake			X**
	Northern Basin & Range		X	

* For watersheds greater than 20 mi^2 in the southern desert regions, both the 6-hour Convective Storm (AMC I) and the 24-hour General Storm (AMC II) should be analyzed and the larger of the two peak discharges selected.

** The use of regional regression equations is recommended where streamgage data are not available; otherwise, hydrologic modeling could be performed with snowmelt simulation.

Table 819.7I

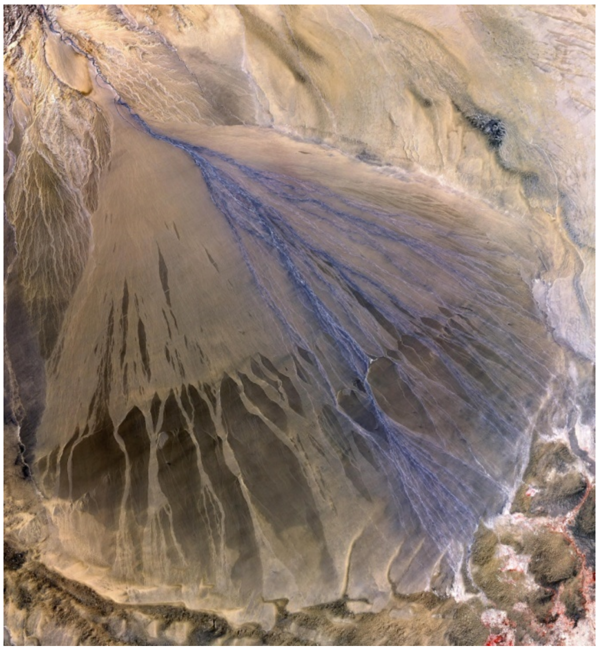
Bulking Factors & Types of Sediment Flow

Sediment Flow Type	Bulking Factor	Sediment Concentration by Weight	Sediment Concentration by Volume
		(100% by WT = 1 x 10^6 ppm)	(specific gravity = 2.65)
	0	0	0
Normal Streamflow	1.11	23	10
	1.25	40	20
Hyperconcentrated Flow	1.43	52	30
	1.67	53	40
Debris Flow	2.00	72	50
Landslide	2.50	80	60
	3.33	87	70

source which is a site visit. An example of an alluvial fan, shown in plan view, is in Figure 819.7F and Figure 872.3.

Figure 819.7F

Alluvial Fan



(e) Wildfire and Debris Flow

After fires have impacted a watershed, sediment/debris flows are caused by surface erosion from rainfall runoff and landsliding due to rainfall infiltration into the soil. The most dominant cause is the runoff process because fire generally reduces the infiltration and storage capacity of soils, which increases runoff and erosion.

1. Fire Impacts

Arid regions do not have the same density of trees and vegetation as a forested area, but the arid environment still falls victim to fires in a similar manner. Prior to a fire, the arid region floor can contain a litter layer (leaves, needles, fine twigs, etc.), as well as a duff layer (partially decomposed components of the litter layer). These layers absorb water, provide storage of rainfall, and protect hillsides. Once these layers are burned, they become ash and charcoal particles that seal soil pores and decrease infiltration potential of the soil, which ultimately increases runoff and erosion.

In order to measure the burn severity of watersheds with respect to hydrologic function, classes of burn severity have been created. These classes are simply stated as high, moderate, low, and unburned. From moderate and high burn severity slopes, the generated sediment can reach channels and streams causing bulked water flows during storm events. Generally speaking, the denser the vegetation in a watershed prior to a fire and the longer a fire burns within this watershed, the greater the effects on soil hydrologic function. This occurs due to the fire creating a water repellent layer at or near the soil surface, the loss of soil structural stability, which all results in more runoff and erosion. After a one or two-year period, the water repellent layer is usually washed away.

(f) Local Agency Methods For Predicting Bulking Factors

1. San Bernardino County

Instead of conducting a detailed analysis, San Bernardino Flood Control District uses a set value for bulking of 2 (i.e., 100 percent bulking) for any project where bulking flows may be anticipated. This bulking factor of 2 can also be expressed as a 50 percent sediment concentration by volume, which is about the upper limit of debris flow. A higher percentage of sediment concentration would be considered a landslide instead of debris flow. Basically, the San Bernardino County method assumes debris flow conditions for all types of potential bulking.

2. Los Angeles County

The Los Angeles (LA) County method uses a watershed-specific bulking factor. The LA County Sedimentation

Manual, which is located at http://ladpw.org/wrd/publication, divides the county into three basins: LA Basin, Santa Clara River Basin, and Antelope Valley, where only the latter is located in the Caltrans desert hydrology regions. The production of sediment from these basins is dependent upon many factors, including rainfall intensity, vegetative cover, and watershed slope. For each of the LA County basins, Debris Potential Area (DPA) zones have been identified.

The Design Debris Event (DDE) is associated with the 50-year, 24-hour duration storm, and produces the quantity of sediment from a saturated watershed that is recovered from a burn. For example, a DPA 1 zone sediment rate of 120,000 cubic yards per square mile has been established as the DDE for a 1-square mile drainage area. This sediment rate is recommended for areas of high relief and granitic formation found in the San Gabriel Mountains. In other mountainous areas in LA County, lower sediment rates have been assigned based on differences in topography, geology, and precipitation. For the Antelope Valley basin, eight debris production curves have been generated, and can be found in Appendix B of the LA County Sedimentation Manual along with curves for the other basins.

In addition to sediment production rates, a series of peak bulking factor curves are presented for each LA County basin in Appendix B of the LA manual. The peak bulking factor can be estimated using these curves based on the watershed area and the DPA. Within the Antelope Valley basin, maximum peak bulking factors range from 1.2 in DPA Zone 11 to 2.00 in DPA Zone 1.

3. Riverside County

For Riverside County, a bulking factor is calculated by estimating a sediment/debris yield rate for a specific storm event, and relating it to the largest expected sediment yield of 120,000 cubic yards per square mile for a 1-square mile watershed from the LA County procedure. This sediment rate from LA County is based on the DPA Zone 1 corresponding to the highest expected bulking factor of 2.00.

The bulking factor equation from the Riverside County Hydrology Manual (http://www.floodcontrol.co.riverside.ca.us/downloads/planning/) is as follows:

$$BF = 1 + \frac{D}{120{,}000}$$

BF = Bulking Factor

D = Design Storm Sediment/Debris Production Rate For Study Watershed (cubic yards/square mile)

4. U.S. Army Corps of Engineers- LA District

This method, located at http://www.spl.usace.army.mil/resreg/htdocs/Publications.html, was originally developed to calculate unit sediment/debris yield values for an “n-year” flood event, and applied to the design and analysis of debris catching structures in coastal Southern California watersheds. The LA District method considers frequency of wildfires and flood magnitude in its calculation of unit debris yield. Even though its original application was intended for coastal-draining watersheds, this method can also be used for desert-draining watersheds for the same local mountain ranges.

The LA District method can be applied to watershed areas between 0.1 and 200 mi^2 that have a high proportion of their total area in steep, mountainous topography. This method is best used for watersheds that have received significant antecedent rainfall of at least 2 inches in 48 hours. Given this criteria, the LA District method is more suited for general storms rather than thunderstorms.

As shown below, this method specifies a few equations to estimate unit debris yield dependent upon the areal size of the watershed. These equations were developed by multiple regression analysis using known sediment/debris data.

For watersheds between 3 and 10 mi^2, the following equations can be used:

$$\log Dy = 0.85 \log Q + 0.53 \log RR$$
$$+0.04 \log A + 0.22FF$$

D_y = Unit Debris Yield (cubic yards/square mile)

RR = Relief Ratio (foot/mile), which is the difference in elevation between the highest and lowest points on the longest watercourse divided by the length of the longest watercourse

A = Drainage Area (acres)

FF = Fire Factor

Q = Unit Peak Runoff (cfs/square mile)

In order to account for increase in debris yield due to fire, a non-dimensional fire factor (FF) is a component in the equation above. The FF varies from 3.0 to 6.5, with a higher factor indicating a more recent fire and more debris yield. This factor is 3.0 for desert watersheds because the threat and effects from fire are minimal.

Because the data used to develop the regression equation was taken from the San Gabriel Mountains, an Adjustment and Transposition (A-T) factor needs to be applied to debris yields from the study watersheds. The A-T factor can be determined using Table 819.7J by finding the appropriate subfactor for each of the four groups (Parent Material, Soils, Channel Morphology, and Hillside Morphology) and summing the subfactors. This sum is the total A-T factor, and it must be multiplied by the sediment/debris yield.

Once the sediment/debris yield value has been determined based on the unit yield, a bulking factor can be calculated using a series of equations. The first equation provides a translation of the clear-water discharge to a sediment discharge. This clear-water discharge should be developed using a hydrograph method and a hydrologic modeling program, such as HEC-HMS.

$$Q_S = aQ_W^n$$

Q_S = Sediment Discharge (cfs)

Q_W = 100-Year Clear-Water Discharge (cfs)

a = Bulking Constant

For a majority of sand-bed streams, the value of "n" is between 2 and 3. When n=2, the bulking factor is linearly proportional to the clear-water discharge. As for the coefficient "a", it is determined with the following equation:

$$a = \frac{V_S}{\Delta t \sum Q_W^2}$$

V_S = Total Sediment Volume (cubic feet)

Δt = Computation Time Interval Used In Developing Hydrograph From Hydrologic Model (e.g. HEC-HMS)

Finally, the bulking factor equation is expressed as follows:

$$BF = \frac{Q_W + Q_S}{Q_W} = 1 + aQ_W^{n-1}$$

(g) Recommended Approach For Developing Bulking Factors

A flow chart outlining the recommended bulking factor process is provided in Figure 819.7H, which considers all bulking methods presented in Topic 819.

As shown in Steps 4 and 5 on Figure 819.7H, a bulking factor can be found by:

a. Identifying the type of flow within a watershed and selecting the corresponding bulking factor, or

b. Using one of the agency methods to calculated the bulking factor.

If the type of flow cannot be identified or the project site does not fall within the recommended boundaries from Figure 819.7H, use the LA District Method because it is the most universal given its use of the Adjustment-Transposition factor based on study watershed properties.

Table 819.7J

Adjustment-Transportation Factor Table

	A-T SUBFACTOR				
	0.25	0.20	0.15	0.10	0.05
PARENT MATERIAL	SUBFACTOR GROUP 1				
Folding	Severe		Moderate		Minor
Faulting	Severe		Moderate		Minor
Fracturing	Severe		Moderate		Minor
Weathering	Severe		Moderate		Minor
SOILS	SUBFACTOR GROUP 2				
Soils	Non-cohesive		Partly Cohesive		Highly Cohesive
Soil Profile	Minimal Soil Profile		Some Soil Profile		Well-developed Soil Profile
Soil Cover	Much Bare Soil in Evidence		Some Bare Soil in Evidence		Little Bare Soil in Evidence
Clay Colloids	Few Clay Colloids		Some Clay Colloids		Many Clay Colloids
CHANNEL MORPHOLOGY	SUBFACTOR GROUP 3				
Bedrock Exposures	Few Segments in Bedrock		Some Segments in Bedrock		Many Segments in Bedrock
Bank Erosion	> 30% of Banks Eroding		10 – 30% of Banks Eroding		< 10% of Banks Eroding
Bed and Bank Materials	Non-cohesive Bed and Banks		Partly Cohesive Bed and Banks		Mildly Cohesive Bed and Banks
Vegetation	Poorly Vegetated		Some Vegetation		Much Vegetation
Headcutting	Many Headcuts		Few Headcuts		No Headcutting
HILLSLOPE MORPHOLOGY	SUBFACTOR GROUP 4				
Rills and Gullies	Many and Active		Some Signs		Few Signs
Mass Movement	Many Scars Evident		Few Signs Evident		No Signs Evident
Debris Deposits	Many Eroding Deposits		Some Eroding Deposits		Few Eroding Deposits
The A-T Factor is the sum of the A-T Subfactors from all 4 Subfactor Groups.					

CHAPTER 830 TRANSPORTATION FACILITY DRAINAGE

Topic 831 - General

Index 831.1 - Basic Concepts

Roadway drainage involves the collection, conveyance, removal, and disposal of surface water runoff from the traveled way, shoulders, sidewalks, and adjoining areas defined in Index 62.1(7) as comprising the roadway. Roadway drainage is also concerned with the handling of water from the following additional sources:

- Surface water from outside the right of way and not confined to channels that would reach the traveled way if not intercepted.
- Crossroads or streets.
- Irrigation of landscaped areas.

The design of roadway drainage systems often involves consideration of the problems associated with inadequate drainage of the adjacent or surrounding area. Cooperative drainage improvement projects with the responsible local agency may offer the best overall solution. Cooperative agreements are more fully discussed under Index 803.2

Some of the major considerations of good roadway drainage design are:

- Facility user safety.
- Convenience to vehicular, bicycle and pedestrian traffic.
- Aesthetics.
- Flooding of the transportation facility and adjacent property.
- Subgrade infiltration.
- Potential erosion, pollution and other environmental concerns.
- Economy of construction.
- Economy of maintenance.

This section involves the hydraulic design fundamentals necessary for properly sizing and locating standard highway drainage features such as:

- Asphalt dikes and gutters.
- Concrete curbs and gutters.
- Median drains.
- Roadside ditches
- Overside drains.
- Drop inlets.
- Storm drains.

Removal of storm water from highway pavement surfaces and median areas is more fully discussed in FHWA Hydraulic Engineering Circular No. 22, "Urban Drainage Design Manual". HEC 22 includes discussion of the effects of roadway geometry on pavement drainage; the philosophy of design storm frequency and design spread selection; storm runoff estimating methods; pavement and bridge deck inlets; and flow in gutters. Charts and procedures are provided for the hydraulic analysis and design of roadway drainage features.

831.2 Highway Grade Line

In flat terrain, roadway drainage considerations often control the longitudinal grade line of the highway. A grade line that assures the desirable goal of keeping the traveled way free of flooding can usually be established for new freeway projects and rural conventional highways.

For multilane urban highways with nearly continuous dike or curb along the shoulder or parking area, it is seldom practical to design the highway with a gutter section which will contain all of the runoff even from frequent rains. For this reason the gutter and shoulder combination, and often partial or full width of the traveled way, are used to convey the runoff to inlets.

831.3 Design Storm and Water Spread

Before the hydraulic adequacy of roadway drainage facilities can be analyzed, the quantity of water (design Q) that the facility may reasonably be expected to convey must be estimated. The most important, and often the most difficult phase of this task is the selection of an appropriate design storm frequency for the specific project, location or site under consideration. In order for a design frequency to be meaningful criteria for roadway drainage design, it must be tied to an acceptable tolerance of flooding. Design water spread, encroachment upon the roadbed or adjacent property, is the tolerance of flooding directly related to roadway drainage design. Allowing too little spread is uneconomical in design and too much spread may result in unsafe driving conditions.

To optimize economy in roadway drainage, the allowable water spread should vary, depending on the type of project being designed. Because of the effect of splash and spray on motorist visibility and vehicle control, high volume roads with high speed traffic cannot tolerate as much water spread as urban streets. Likewise, the allowable water spread should be minimized on urban streets where a large number of pedestrians use adjacent sidewalks and pedestrian crosswalks. Consideration should be given to the element of motorist surprise when encountering intermittent puddles rather than a continuous encroachment of water on the driving lane. Eccentric forces are exerted on a vehicle when one side encounters water in the lane and the other side does not.

The probability of exceedance of the design storm and the acceptable tolerance to flooding depends on the importance of the highway and risks involved. Selection of the design storm and water spread parameters on rehabilitation and reconstruction are generally controlled by existing constraints.

In addition to the major roadway drainage considerations previously listed, the following more specific factors are to be considered in establishing the project design storm:

- Highway type
- Traffic volume
- Design speed
- Local standards

The following geometric and design features of the highway directly affect establishment of the project design water spread:

- Cross slope
- Longitudinal slope
- Number of lanes
- Width of shoulders
- Height of curb and dike
- Parking lanes
- Bus/Transit pullouts and loading areas

Desirable limits for water spread with respect to design storm probability of exceedance are given in Table 831.3. The parameters shown are considered minimum roadway drainage design standards for new freeway construction and for all State highways with depressed sections which require pumping. Local conditions may justify less stringent criteria than the table parameters for conventional highways. Exceptions should be documented by memo to the project file.

It is often advantageous, to both the State and the local agency, for highway drainage and street drainage to be compatible. This is particularly true in urban areas and rapidly developing suburban areas where a conventional highway is, or will become, part of the street network. Street drainage criteria adopted by a local agency are generally based on the hydrologic events peculiar to a geographical area. Local drainage standards that satisfy the needs of the community, usually provide reasonable traffic safety and flood risk considerations commensurate with those normally expected for conventional highways in urban areas.

831.4 Other Considerations

(1) Sheet Flow. Concentrations of sheet flow across roadways are to be avoided. As a general rule, no more than 0.10 cubic feet per second should be allowed to concentrate and flow across a roadway. Particular attention should be given to reversal points of superelevation where shoulder and gutter slopes may direct flows across the roadway and gore areas.

(2) Stage Construction. All permanent features of roadway drainage systems should be designed and constructed for the ultimate highway facility.

Table 831.3

Desirable Roadway Drainage Guidelines

HIGHWAY Type/Category/Feature	DESIGN STORM		DESIGN WATER SPREAD		
	4% (25 yrs)	10% (10 yrs)	Shldr or Parking Lane	1/2 Outer Lane	Local Standard
FREEWAYS					
Through traffic lanes, branch connections, and other major ramp connections.	X	--	X	--	--
Minor ramps.	--	X	X	--	--
Frontage roads.	--	X	--	--	X
CONVENTIONAL HIGHWAYS					
High volume, multilane Speeds over 45 mph.	X	--	X	--	--
High volume, multilane Speeds 45 mph and under.	--	X	--	X	--
Low volume, rural Speeds over 45 mph.	X	--	X	--	--
Urban Speeds 45 mph and under.	--	X	--	--	X
ALL STATE HIGHWAYS					
Depressed Sections That Require Pumping:					

Use a 2% (50 yrs) design storm for freeways and conventional State highways. Design water spread at depressed sections should not exceed that of adjacent roadway sections. A 4% (25 yr) design storm may be used on local streets or road undercrossings that require pumping.

(3) *Landscaping.* Runoff from existing or proposed landscaping, including excess irrigation water runoff, must be considered.

(4) *Groundwater.* Groundwater is subsurface water within a permeable strata. Depending upon recharge and withdrawal rates the level of the groundwater table can fluctuate greatly, over a period of a few months or over periods of many years. Consideration should be given to recent history (several years of abnormally wet or dry conditions) as well as the possibility of revised practices by local water districts (either increased pumping or increased recharge).

Pipes located in areas where contact with groundwater within their design life is likely should have watertight joints. If groundwater contact is likely and the surrounding soils are highly erodible (fine grained sand, silty sand and sandy silt/silt of limited cohesion) consideration should be given to wrapping the pipe joint with filter fabric. The fabric should cover a length of 4 feet along the pipe, centered on the joint. Groundwater at or above the drainage system elevation will lead to infiltration. Where this is undesirable, either joint systems capable of resisting the hydrostatic pressure, or dewatering measures, should be incorporated into the design. The design of groundwater control measures must be coordinated with Geotechnical Services in the Division of Engineering Services.

(5) *Hydroplaning.* Hydroplaning is the separation of the tire from the road surface by a thin layer of liquid (usually water) on the pavement. The liquid separates the tire from the pavement because of viscosity (viscous hydroplaning), dynamic lift (dynamic hydroplaning), or a combination of the two. Since water offers little shear resistance, the tire loses its tractive ability and the driver has a loss of control of the vehicle. At locations where there is a potential for hydroplaning, a careful review of the wet weather accident rates should be made using information obtained from the District Traffic Branch. Typical situations that should be evaluated for hydroplaning potential are:

* Where three (3) lanes or more are sloped in the same direction (see Topic 833).
* Where the longitudinal grade and or cross slope are less than minimum (Refer to Index 204.3 for minimum grade and Indexes 301.2 and 302.2 for cross slope).
* Where there are poor pavement conditions (rutting, depressions, inadequate roughness).
* Where water is allowed to concentrate prior to being directed across the travel lanes (see Index 831.4(1)).
* Where re-striping projects will reduce shoulder widths where dike, curb or concrete barrier are present.

These situations may also be present on median widening projects or projects involving pavement rehabilitation and or lane addition on multi-lane highways or freeways.

Speed and tire pressure appear to be a significant factors in the occurrence of hydroplaning, therefore, it is considered to be the driver's responsibility to exercise prudence and caution when driving during wet conditions (California Basic Speed Law).

Designers do not have control over all of the factors involved in hydroplaning. However, remedial measures may be included in development of a project to reduce hydroplaning potential. The following is provided as guidance for the designer as practical measures to consider:

(1) Pavement Sheet Flow

- Maximize transverse slope (see Topic 833)
- Maximize pavement roughness
- Use of graded course (porous pavements)

(2) Gutter Flow

- Limit water spread to Table 831.3
- Maximize interception of gutter flow above superelevation transitions (see Index 837.3)

(3) Sag Areas

- Limit pond duration and depth (see Topic 833)

(4) Overtopping

- Avoid overtopping at cross culverts using appropriate freeboard and/or headwater elevation (see Topic 821)

Where suitable measures cannot be implemented to address conditions such as those identified above, or an identified existing problem area, coordination should be made with the Safety Review Committee per Index 110.7.

831.5 Computer Programs

There are many computer programs available to aid highway design engineers with estimating runoff and ensuing hydraulic design and analysis of roadway drainage facilities.

Refer to Table 808.1 for guidance on selecting appropriate software programs for specific analysis needs.

Familiarity with the fundamentals of hydraulics and traditional methods of solution are necessary to assure that the results obtained are reasonable. There is a tendency for inexperienced engineers to accept computer output as valid without verifying the reasonableness of input and output data.

Topic 832 - Hydrology

832.1 Introduction

The philosophy and principles of hydrology are discussed in Chapter 810. Additional information on methods of estimating storm runoff may be found in FHWA's HEC 22.

832.2 Rational Method

With few exceptions, runoff estimates for roadway drainage design are made by using Rational Methods described under Index 819.2(1). In order to make use of these methods, information on the intensity, duration, and frequency of rainfall for the locality of the project must be established. Refer to Index 815.3(3) for further information on precipitation intensity-duration-frequency (IDF) curves that have been developed for many locations in California.

832.3 Time of Concentration

Refer to Index 816.6 for information on time of concentration.

Topic 833 - Roadway Cross Sections

833.1 Introduction

The geometric cross section of the roadway affects drainage features and hydraulic considerations. Cross slope and width of pavement and shoulders as well as other roadway geometry affect the rate of runoff, width of tolerable spread, and hydraulic design considerations. The cross section of drainage features such as, depressed medians, curbs and gutters, dikes, and side ditches is often controlled by an existing roadway geometric cross section or the one selected for new highway construction.

833.2 Grade, Cross Slope and Superelevation

The longitudinal slope or grade is governed by the highway grade line as discussed under Index 831.2. Refer to Index 204.3 for minimum grade and Indexes 301.2 and 302.2 for cross slope. Where three (3) lanes or more are sloped in the same direction, it is desirable to counter the resulting increase in flow depth by increasing the cross slope of the outermost lanes. The two (2) lanes adjacent to the crown line should be pitched at the normal slope, and successive lane pairs, or portions thereof outward, should be increased by about 0.5 to 1 percent. The maximum pavement cross slope should be limited to 4 percent. However, exceptions to the design criteria for cross slope in Index 302.2 must be formally approved in accordance with the requirements Index 82.2, "Approvals for Nonstandard Design." For projects where lanes will be added on the inside of divided highways, or when widening an existing “crowned” 2-lane highway to a 4-lane divided highway, consideration should be given to the use of a “tent section” in order to minimize the number of lanes sloping in the same direction. Refer to Index 301.2. Consideration should be given to increasing cross slopes in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades. Superelevation is discussed in Topic 202. Refer to Index 831.4 for Hydroplaning considerations.

Topic 834 - Roadside Drainage

834.1 General

Median drainage, ditches and gutters, and overside drains are some of the major roadside drainage facilities.

834.2 Median Drainage

(1) *Drainage Across the Median.* When it is necessary for sheet flow to cross flush medians, it should be intercepted by the use of slotted drains or other suitable alternative facilities. See Standard Plan D98-B for slotted drain details.

Where floodwaters are allowed to cross medians, designers must consider the impacts of railings, barrier or other obstructions to both the depth and spread of flow. Designers should consult their district hydraulic unit for assistance.

(2) *Grade and Cross Slope.* The longitudinal slope or grade for median drainage is governed by the highway grade line as discussed under Index 831.2. Refer to Index 204.3 for minimum grade and Indexes 305.2 and 405.5(4) for standards governing allowable cross slope of medians.

Existing conditions control median grades and attainable cross slope on rehabilitation projects. The flattest desirable grade for earth medians is 0.25 percent and 0.12 percent for paved gutters in the median.

(3) *Erosion.* When velocities are excessive for soil conditions, provisions for erosion control should be provided. See Table 865.2 for recommended permissible velocities for unlined channels.

Economics and aesthetics are to be taken into consideration in the selection of median erosion control measures. Under the less severe conditions, ground covers of natural or synthetic materials which render the soil surface stable against accelerated erosion are adequate. Under the more severe conditions, asphalt or concrete ditch paving may be required.

Whenever median ditch paving is necessary, consideration should be given to the use of cement or lime treatment of the soil. The width treated will depend on the capacity needed to handle the drainage. A depth of 6 inches is generally satisfactory. The amount of cement or lime to be used should be based on laboratory tests of the in-place material to be tested, and normally varies from 6 percent to 10 percent. If a clear or translucent curing compound is used, the completed area is unobtrusive and aesthetically pleasing.

Asphalt concrete ditch paving and soil cement treatments cured with an application of liquid asphalt are highly visible and tend to become unsightly from streaks of eroded material. Cobbles, though effective for erosion control, are not satisfactory in a recovery area for out of control vehicles. See Topic 872 for further discussion on erosion protection and additional types of ditch linings. Erosion control references are given under Index 871.3.

(4) *Economy in Design.* Economy in median drainage can be achieved by locating inlets to utilize available nearby culverts or the collector system of a roadway drainage installation. The inlet capacity can be increased by placing it in a local depression. Use of slotted pipe at sag points where a local depression might be necessary may be an alternative solution to a grate catch basin.

834.3 Ditches and Gutters

(1) *Grade.* The flattest grade recommended for design is 0.25 percent for earth ditches and 0.12 percent for paved ditches.

(2) *Slope Ditches.* Slope ditches, sometimes called surface, brow, interception, or slope protection ditches, should be provided at the tops of cuts where it is necessary to intercept drainage from natural slopes inclined toward the highway.

When the grade of a slope ditch is steep enough that erosion would occur, the ditch should be paved. Refer to Table 865.2 for permissible velocities for unlined channels in various types of soil. When the ditch grade exceeds a 4:1 slope, a downdrain is advisable. Slope ditches may not be necessary where side slopes in favorable soils are flatter than 2:1 or where positive erosion control measures are to be instituted during construction.

(3) *Side Gutters.* These are triangular gutters adjoining the shoulder as shown in Figures 307.2 and 307.5. The main purpose of the 3 feet wide side gutter is to prevent runoff from the cut slopes on the high side of superelevation from flowing across the roadbeds. The use of side gutters in tangent alignment should be avoided where possible. Local drainage conditions, such as in snow areas, may require their use on either tangent or curved alignment in cut sections. In snow areas it may be necessary to increase the width of side gutters from 3 feet to 6 feet. The slope from the edge of the shoulder to the bottom of the gutter should be no steeper than 6:1. The structural section for paved side gutters should be adequate to support maintenance equipment loads.

(4) *Dikes.* Dikes placed adjoining the shoulder, as shown in Figures 307.2, 307.4, and 307.5, provide a paved triangular gutter within the shoulder area. For conditions governing their use, see Index 303.3.

(5) *Chart Solutions.* Charts for solutions to triangular channel flow problems are contained in FHWA Hydraulic Engineering Circular No. 22, "Urban Drainage Design Manual".

834.4 Overside Drains

The purpose of overside drains, sometimes called slope drains, is to protect slopes against erosion. They convey down the slope drainage which is collected from the roadbed, the tops of cuts, or from benches in cut or fill slopes. They may be pipes, flumes or paved spillways.

(1) *Spacing and Location.* The spacing and location of overside drains depend on the configuration of the ground, the highway profile, the quantity of flow and the limitations on flooding stated in Table 831.3. When possible, overside drains should be positioned at the lower end of cut sections. Diversion from one watershed to another should be avoided. If diversion becomes necessary, care should be used in the manner in which this diverted water is disposed.

Overside drains which would be conspicuous or placed in landscaped areas should be concealed by burial or other means.

(2) *Type and Requirement.* Following are details of various types of overside drains and requirements for their use:

(a) Pipe Downdrains. Metal and plastic pipes are adaptable to any slope. They should be used where side slopes are 4:1 or steeper. Long pipe downdrains should be anchored.

The minimum pipe diameter is 8 inches but large flows, debris, or long pipe installations may dictate a larger diameter.

Watertight joints are necessary to prevent leakage which causes slope erosion. Economy in long, high capacity downdrains is achieved by using a pipe taper in the initial reach. Pipe tapers should insure improved flow characteristics and permit use of a smaller diameter pipe below the taper. See Standard Plan D87-A for details.

(b) Flume Downdrains. These are rectangular corrugated metal flumes with a tapered entrance. See the Standard Plan D87-D for details. They are best adapted to slopes that are 2:1 or flatter but if used on 1.5:1 slopes, lengths over 60 feet are not recommended. Abrupt changes in alignment or grade should be avoided. Flume downdrains should be depressed so that the top of the flume is flush with the fill slope.

(c) Paved Spillways. Permanent paved spillways should only be used when the side slopes are flatter than 4:1. On steeper slopes a more positive type of overside drain such as a pipe downdrain should be used.

Temporary paved spillways are effective in preserving raw fill slopes that are 6:1 or flatter in friable soils during the period when protective growth is being established. Paved spillways should be spaced so that a dike 2 inches high placed at the outer edge of the paved shoulder will effectively confine drainage between spillways. When it is necessary to place a spillway on curved alignment, attention must be given to possible overtopping at the bends. See Index 868.2(3) for discussion of superelevation of the water surface.

(3) *Entrance Standards.* Entrance tapers for pipes and flume downdrains are detailed on the Standard Plans. Pipe entrance tapers should be depressed at least 6 inches.

The local depressions called "paved gutter flares" on the Standard Plans are to be used at all entrance tapers. See Standard Plans D87-A and D87-D for details and Index 837.5 for further discussion on local depressions.

In areas where local depressions would decrease safety the use of flush grate inlets or short sections of slotted drain for entrance structures may be necessary.

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

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

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

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

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

Topic 835 - Dikes and Berms

835.1 General

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

835.2 Earth Berms

(Text Later)

835.3 Dikes

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

Topic 836 - Curbs and Gutters

836.1 General

The primary reason for constructing curbs and gutters may be for delineation or pedestrian traffic rather than for drainage considerations. Refer to Topic 303 for further discussion and Standard Plan A87 for details on concrete curbs and gutters.

Whatever the justification for constructing curbs and gutters, they will usually have an effect on surface water runoff and result in becoming a roadway drainage design consideration.

836.2 Gutter Design

(1) *Capacity.* Gutters and drainage facilities are to be designed to keep flooding within the limits given in Table 831.3. Easy solutions to gutter flow problems can be obtained by using the charts contained in FHWA Hydraulic Engineering Circular No. 22, "Urban Drainage Design Manual" which applies to triangular channels and other shapes illustrated in the charts. Parked cars reduce gutter capacity and also can cause water to shoot over the curb. The downstream ends of driveway ramps can also cause water to flow over the curb. As a rule of thumb, gutter capacity should be determined on a depth equal to 0.5 the curb height for grades up to 10 percent and 0.4 the curb height for grades over 10 percent in locations where parking is allowed or where driveways are constructed.

(2) *Grade and Cross Slope.* The longitudinal grade of curbs and gutters is controlled by the highway grade line as discussed under Index 831.2.

The cross slope of standard gutters is typically 8.33 percent toward the curb. Pavement slopes on superelevated roadways extend the full width of the gutter, except that gutter slopes on the low side should be not less than 8.33 percent. Because they cut down gutter capacity and severely reduce inlet efficiency, cross slopes flatter than 8.33 percent should be avoided, except where gutters are adjacent to curb ramps where ADA requirements limit the slope to a maximum of 5 percent.

(3) *Curbed Intersections.* If pedestrian traffic is a ruling factor, intersection drainage presents the following alternatives to be weighed as to effectiveness and economy.

(a) Intercept the whole flow upstream of the crosswalk.

(b) Intercept a part of the water and allow the overflow to cross the intersection. The width of flow should be controlled so that pedestrian traffic is not unduly hampered.

(c) If flow is small, pass the entire flow across the intersecting street in a valley gutter.

(4) *Valley Gutters.* Valley gutters across the traveled way of the highway should not be used. Valley gutters may be used across intersecting streets and driveways, however, at intersections with high traffic volumes on all approaches, it is desirable to intercept all gutter flow upstream of the intersection and avoid the use of valley gutters. Valley gutters are also undesirable along streets where speeds are relatively high. In locations of frequent intermittent low flows, the use of valley gutters with slotted drains should be considered. In general, the total width of gutters should not exceed 6 feet and cross slopes should not exceed 3 percent. Two percent is suggested where more than nominal speeds are involved.

Topic 837 - Inlet Design

837.1 General

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

837.2 Inlet Types

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

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

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

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

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

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

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

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

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

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

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

(a) Type GO and GDO. These types of inlets have a curb opening directly opposite the grate. The GDO inlet has two grates placed side by side and is designed for intercepting a wide flow. A typical use of these inlets would be in a sag location either in a curb and gutter installation or within a shoulder fringed by a dike. When used as the surface inlet for a pumping installation, the trash rack shown on the Standard Plan D74B is provided.

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

(4) *Pipe.* Pipe drop inlets are made of a commercial pipe section of concrete or corrugated metal. As a class, they develop a high capacity and are generally the most economical type. This type of inlet is intended for uses outside the roadbed at locations that

will not be subjected to normal highway wheel loads.

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

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

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

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

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

(6) Grated Line Drains. This type of inlet is made of monolithic polymer concrete with a ductile iron frame and grate on top. This type of inlet can be used as an alternative at the locations described under slotted drains, preferably in shoulder areas away from traffic loading. However, additional locations may include localized flat areas of pavement at private and public intersections, superelevation transitions, along shoulders where widening causes a decrease to allowable water spread, tollbooth approaches, ramp termini, parking lots and on the high side of superelevation in snow and ice country to minimize black ice and sheet flow from snow melt. Removable grates should not be placed where subject to traffic.

Short sections of grated line drain may be used in conjunction with an existing drainage inlet as a supplement in sag locations. However, based on the depth of the water, the flow condition will be either weir or orifice. The transition between weir and orifice occurs at approximately 7 inches depth of flow. The HEC-22 method of design for slotted pipe is recommended as the basis for grated line drain design. It should be noted that this is inlet interception/capacity design, not the carrying capacity of the product as a conduit.

Furthermore, the grated line drain has a smaller cross sectional area than slotted pipe, and therefore typically less carrying capacity.

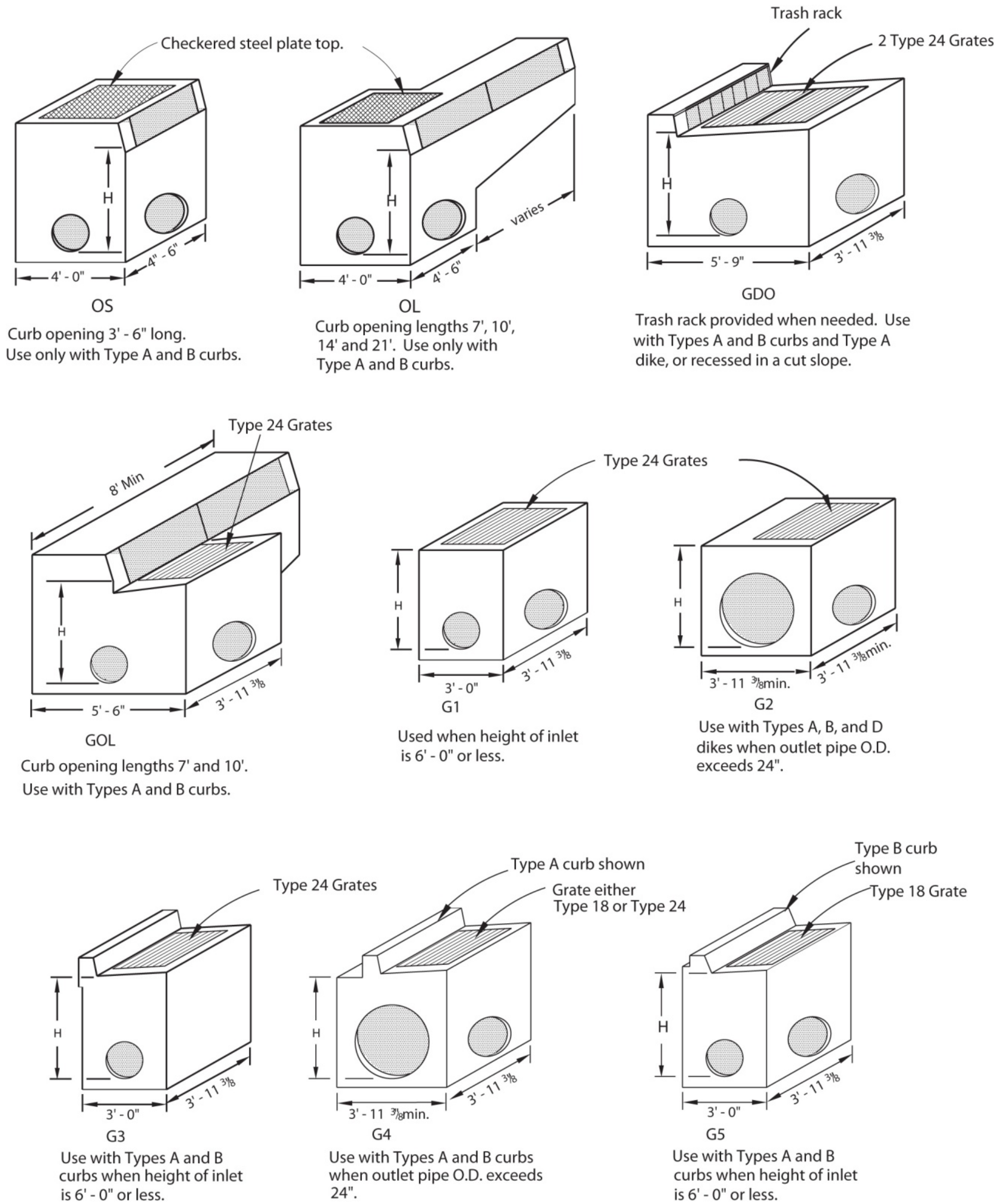
Grated line drains are recommended as an alternative to slotted pipe at locations susceptible to pipe clogging from sediments and debris. Self-cleaning velocities can usually be generated from their smooth interior surface, or if necessary by specifying the optional pre-sloped sections.

Grated line drains may also be useful where there is a potential for utility conflicts with slotted drains, which are generally installed at a greater depth.

At locations where clean out access is needed, removable grates can be specified. In areas with pedestrian traffic, special grates which meet the Americans with Disabilities Act (ADA) requirements are mandatory. This type of grate is susceptible to clogging, therefore removable grates are recommended at these locations, and they should only be specified when placement directly within the pedestrian path of travel is unavoidable.

(7) Scuppers. This type of inlet consists of a low, rectangular slot cut through the base of a barrier. Similar to, but smaller than curb opening inlets (See Index 837.2(1)), scuppers

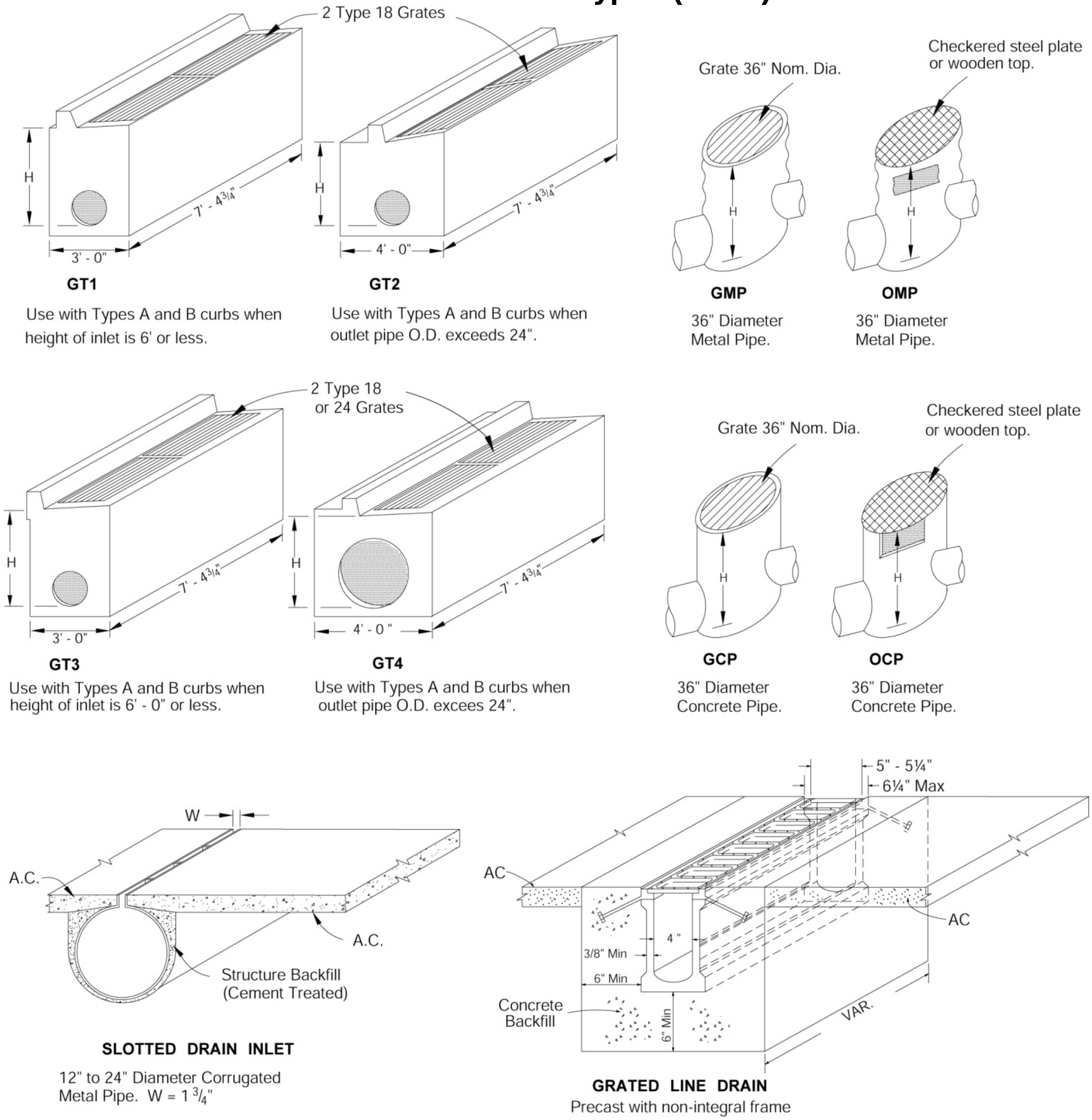
Figure 837.1
Storm Drain Inlet Types



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

Figure 837.1

Storm Drain Inlet Types (Cont.)



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

are prone to clogging by sediment and debris and require enhanced maintenance attention. Scupper interception efficiency decreases with increased longitudinal gradient and scupper design is not typically compatible with construction of an inlet depression. Scuppers are typically considered only when other inlet options are infeasible.

837.3 Location and Spacing

(1) *Governing Factors.* The location and spacing of inlets depend mainly on these factors:

(a) The amount of runoff,

(b) The longitudinal grade and cross slope,

(c) The location and geometrics of interchanges and at-grade intersections,

(d) Tolerable water spread, see Table 831.3,

(e) The inlet capacity,

(f) Accessibility for maintenance and inspection,

(g) Volume and movements of motor vehicles, bicycles and pedestrians,

(h) Amount of debris, and

(i) The locations of public transit stops.

(2) *Location.* There are no ready rules by which the spacing of inlets can be fixed; the most effective and economical installation should be the aim.

The following are locations where an inlet is nearly always required:

- Sag points
- Points of superelevation reversal
- Upstream of ramp gores
- Upstream and downstream of bridges – bridge drainage design procedure assumes no flow onto bridge from approach roadway, and flow off bridge to be handled by the district.
- Intersections
- Upstream of pedestrian crosswalks
- Upstream of curbed median openings

In urban areas, the volume and movements of vehicles, bicyclists, and pedestrians constitute an important control. For street or road crossings, the usual inlet location is at the intersection at the upstream end of the curb or pavement return and clear of the pedestrian crosswalk. Where the gutter flow is small and vehicular, bicycle, and pedestrian traffic are not important considerations, the flow may be carried across the intersection in a valley gutter and intercepted by an inlet placed downstream. See Index 836.2(4).

At depressed grade lines under structures, care must be taken to avoid bridge pier footings. See Index 204.6.

Safety of location for maintenance purposes is an important consideration. Wall opening inlets should not be placed where they present an obstacle to maintenance equipment and to vehicles that leave the traveled way. Grate top inlets should be installed in such locations.

Placement of inlets within the traveled way is discouraged. Inlets should typically be relocated when roadways are widened or realigned. Any proposal to leave an existing or construct a new inlet within the traveled way should be discussed with District Maintenance to verify that future access is feasible.

(3) *Spacing.* Arbitrary spacing of inlets should be avoided. The distance between inlets should be determined by a rational analysis of the factors mentioned above. Detailed procedures for determining inlet spacing are given in FHWA Hydraulic Engineering Circular No. 22, "Urban Drainage Design Manual". In a valley median, the designer should consider the effect of inlet spacing on flow velocities where the soil is susceptible to erosion. To economize on disposal facilities, inlets are often located at culverts or near roadway drainage conduits.

(4) *Inlets in Series.* Where conditions dictate the need for a series of inlets, the recommended minimum spacing should be approximately 20 feet to allow the bypass flow to return to the curb face.

837.4 Hydraulic Design

(1) *Factors Governing Inlet Capacity.* Inlet capacity is a variable which depends on:

(a) The size and geometry of the intake opening,

(b) The velocity and depth of flow and the gutter cross slope just upstream from the intake, and

(c) The amount of depression of the intake opening below the flow line of the waterway.

(2) *General Notes.*

(a) Effect of Grade Profile. The grade profile affects both the inlet location and its capacity. The gutter grade line exerts such an influence that it often dictates the choice of inlet types as well as the gutter treatment opposite the opening. See Index 831.2.

Sag vertical curves produce a flattening grade line which increases the width of flow at the bottom. To reduce ponding and possible sedimentation problems, the following measures should be considered:

- Reduce the length of vertical curve.
- Use a multiple installation consisting of one inlet at the low point and one or more inlets upstream on each side. Refer to HEC 22 for further discussion and design procedures for locating multiple inlets.

  Short sections of slotted or grated line drains on either side of the low point may be used to supplement drop inlets.

(b) Cross Slope for Curbed Gutters. Make the cross slope as steep as possible within limits stated under Index 836.2(2). This concentrates the flow against the curb and greatly increases inlet capacity.

(c) Local Depressions. Use the maximum depression consistent with site conditions; for further details see Index 837.5.

(d) Trash. The curb-opening type inlet, when the first in a series of grate inlets, may intercept trash and improve grate efficiency. In a grade sag, one trash interceptor should be used on each side of the sump.

(e) Design Water Surface Within the Inlet. The crown of the outlet pipe should be low enough to allow for pipe entrance losses plus a freeboard of 0.75 feet between the design water surface and the opening at the gutter intake. This allows sufficient margin for turbulence losses, and the effects of floating trash.

(f) Inlet Floor. The inlet floor should generally have a substantial slope toward the outlet. In a shallow drain system where conservation of head is essential, or any system where the preservation of a nonsilting velocity is necessary, the half round floor shown on the Standard Plan D74C should be used when a pipe continues through the inlet.

(g) Partial Interception. Economies may be achieved by designing inlets for partial interception with the last one or two inlets in series intercepting the remaining flow. See Hydraulic Engineering Circular No. 22.

(3) *Curb-Opening Inlets.* Gutter depressions should be used with curb-opening inlets. The standard gutter depressions for curb-opening inlets, shown on Standard Plan D78 are 0.1 foot and 0.25 foot deep.

Curb-opening inlets are most economical and effective if designed and spaced to intercept only 85 to 90 percent of the flow. This provides for an increased flow depth at the curb face.

Figure 4-11, "Comparison of Inlet Interception Capacity, Slope Variable", and Figure 4-12, "Comparison of Inlet Interception Capacity, Flow Rate Variable" of Hydraulic Engineering Circular No. 22 can be used to obtain interception capacities for various longitudinal grades, cross slopes, and gutter depressions. Charts for determining interception capacities under sump conditions are also available in HEC No. 22.

(4) *Grate Inlets.* The grate inlet interception capacity is equal to the sum of the frontal flow (flow over the grate) interception and the side flow interception. The frontal flow interception will constitute the major portion of the grate interception. In general, grate inlets will intercept all of the frontal flow until a velocity is reached at which water begins to splash over the grate. Charts provided in HEC 22 can be used to compute grate interception capacities for the various grates contained therein. Grate depressions will greatly increase inlet capacity.

The HEC 22 charts neglect the effects of debris and clogging on inlet capacity. In some localities inlet clogging from debris is extensive, while in other locations clogging is negligible. Local experience should dictate the magnitude of the clogging factor, if any, to be applied. In the absence of local experience, design clogging factors of 33 percent for freeways and 50 percent for city streets may be assumed.

Grate type inlets are most economical and effective if designed and spaced to intercept only 75 to 80 percent of the gutter flow.

(5) *Combination Inlets.*

(a) Type GO and GDO Inlet. For design purposes, only the capacity of the grates need be considered. The auxiliary curb opening, under normal conditions, offers little or no increase in capacity; but does act as a relief opening should the grate become clogged. Since the grates of Type GDO are side by side, the inlet capacity is the combined capacity of the two grates.

(b) Type GOL Inlet. The interception capacity of this inlet, a curb-opening upstream of a grate, is equal to the sum of the capacities for the two inlets except that the frontal flow and thus interception capacity of the grate is reduced by interception at the curb opening.

(6) *Pipe Drop Inlets.*

(a) Wall Opening Intake. The standard intake opening 2 feet wide and 8 inches to 12 inches deep provides a capacity of approximately 6.0 CFS when the water surface is 1 foot higher than the lip of the opening. Where the flow is from more than one direction, two or more standard openings may be provided. Higher capacity openings larger than standard may be provided but are of a special design.

(b) Grate Intake. The choice between inlets with a round grate (Types GCP and GMP) and those with a rectangular grate (Type G1) hinges largely on hydraulic efficiency. In a waterway where the greatest depth of flow is at the center, both grates are equally effective. In a waterway where the cross slope concentrates the flow on one side of the grate, the rectangular shape is preferred. For rectangular grates, the charts contained in HEC 22 can be used to compute flow intercept. Round grates (Type 36R) with 0.5 foot of depression develop a capacity of 12 CFS to 15 CFS.

837.5 Local Depressions

(1) *Purpose.* A local depression is a paved hollow in the waterway shaped to concentrate and direct the flow into the intake opening and increases the capacity of the inlet. In a gutter bordered by a curb, it is called a gutter depression.

(2) *Requirements.* Local depressions generally consist of a paved apron or transition of a shape which serves the purpose. Local depressions should meet the following requirements:

(a) Valley Medians. In medians on a grade, the depression should extend a minimum of 10 feet upstream, 6 feet downstream and 6 feet laterally, measured from the edge of the opening. In a grade sag, the depression should extend a minimum of 10 feet on all sides. No median local depression, however should be allowed to encroach on the shoulder area.

The normal depth of depression is 4 inches.

(b) Paved Gutter Flares. The local depression which adjoins the outer edge of shoulder at the entrance to overside downdrains and spillways is labeled "paved gutter flare" on Standard Plans D87-A and D87-D. The flow line approaching the inlet is depressed to increase capacity and minimize water spread on the roadbed. Within a flare length of 10 feet the gutter flow line is depressed a minimum of 6 inches at the inlet. Recommended flare lengths for various gutter flow line depression depths are given on the Standard Plans. When conditions warrant, these flare lengths may be exceeded.

Traffic safety should not be compromised for hydraulic efficiency. Any change in the shape of the paved gutter flare that will result in a depression within the shoulder area should not be made. The Type 2 entrance taper and paved gutter flare is intended for use on divided highways where gutter grades exceed 2 percent and flow is in the opposite direction of traffic.

(c) Roadside Gutter and Ditch Locations. Regardless of type of intake, the opening of a drop inlet in a roadside gutter or ditch should be depressed from 4 inches to 6 inches below the flow line of the waterway with 10 feet of paved transition upstream.

(d) Curb and Gutter Depressions. This type of depression is carefully proportioned in length, width, depth, and shape. To best preserve the design shape, construction normally is of concrete. Further requirements for curb and gutter depressions are:

- Length - As shown on Standard Plan D78.
- Width - Normally 4 feet, but for wide flows or a series of closely spaced inlets, 6 feet is authorized.
- Depth - Where traffic considerations govern, the depth commonly used is 0.1 foot. Use the maximum of 0.25 foot wherever feasible at locations where the resulting curb height would not be objectionable.

(e) Type of Pavement. Local depressions outside the roadbed are usually surfaced with asphalt concrete 0.15 foot thick.

(3) General Notes on Design. Except for traffic safety reasons, a local depression is to be provided at every inlet even though the waterway is unpaved. Where the size of intake opening is a question, a depression of maximum depth should be considered before deciding on a larger opening. For traffic reasons, the gutter depression should be omitted in driveways and median curb and gutter installations.

It is permissible to omit gutter depressions at sump inlets where the width of flow does not exceed design water spread.

Topic 838 - Storm Drains

838.1 General

The total drainage system which conveys runoff from roadway areas to a positive outlet including gutters, ditches, inlet structures, and pipe is generally referred to as a storm drain system. In urban areas a highway storm drain often augments an existing or proposed local drainage plan and should be compatible with the local storm drain system.

This section covers the hydraulic design of the pipe or enclosed conduit portion of a storm drain system.

838.2 Design Criteria

To adequately estimate design storm discharges for a storm drain system in urban areas involving street flooding it may be necessary to route flows by using hydrograph methods. Hydrographs are discussed under Index 816.5 and further information on hydrograph methods may be found in Chapters 6 and 7 of HDS No.2, Highway Hydrology.

838.3 Hydraulic Design

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

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

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

838.4 Standards

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

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

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

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

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

Table 838.4
Minimum Pipe Diameter for Storm Drain Systems

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

NOTE:

(1) 18 minimum if wholly or partly under the roadbed.

Specific subjects for special consideration are:

* Bedding and Backfill. Bedding and backfill consideration are discussed under Index 829.2. Maximum height of cover tables are included in Chapter 850 and minimum thickness of cover is given in Table 856.5.

* Roughness Factor. The roughness factor, Manning's n value, generally assumes greater importance for storm drain design than it does for culverts. Suggested Manning's n values for various types of pipe materials are given in Table 852.1.

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

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

838.5 Appurtenant Structures

(1) Manholes.

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

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

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

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

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

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

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

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

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

(2) Junction Structures. A junction structure is an underground chamber used to join two or more conduits, but does not provide direct access from the surface. It is designed to prevent turbulence in the flow by providing a smooth transition. This type of structure is usually needed only where the trunk drain is 42 inches or more in diameter. A standard detail sheet of a junction structure is available for pipes ranging from 42 inches to 84 inches in diameter at the following Office Engineer web site address: http://www.dot.ca.gov/hq/esc/structures_cadd/XS_sheets/Metric/dgn/. The XS sheet reference is XS 4-26. Where required by spacing criteria, a manhole should be used.

(3) *Flap Gates.* When necessary, backflow protection should be provided in the form of flap 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 gate. Where the failure of a flap gate to close would cause serious damage, a manually controlled gate in series should be considered for emergencies.

Topic 839 - Pumping Stations

839.1 General

Drainage disposal by pumping should be avoided where gravity drainage is reasonable. Because pumping installations have high initial cost, maintenance expense, power costs, and the possibility of failure during a storm, large expenditures can be justified for gravity drainage. In some cases, this can be accomplished with long runs of pipe or continuing the depressed grade to a natural low area.

Whenever possible, drainage originating outside the depressed areas should be excluded. District and Division of Structures cooperation is essential in the design of pumping stations, tributary storm drains, and outfall facilities. This is particularly true of submerged outlets, outlets operating under pressure, and outlets of unusual length.

839.2 Pump Type

Horizontal pumps in a dry location are generally specified for ease of access, safety, and standardization of replacement parts.

Only in special cases is stand-by power for pumping plants a viable consideration. All proposals for stand-by power are to be reviewed by and coordinated with the Division of Structures.

839.3 Design Responsibilities

When a pumping station is required, responsibility for design between the District and the Division of Structures is as follows:

(1) *Districts.* The District designs the collector and the outfall facilities leading from the chamber into which the pumps discharge. This applies to outfalls operating under gravity and with a free outlet. Refer to Topic 838.

Details of pumping stations supportive information to be submitted by the District to the Division of Structures is covered under Index 805.8 and Chapter 3-3.1(4) of the Drafting and Plans Manual.

(2) *Division of Structures.* The Division of Structures will prepare the design and contract plans for the pumping station, the storage box and appurtenant equipment, considering the data and recommendations submitted by the District.

The Division of Structures will furnish the District a preliminary plan based on data previously submitted by the District. It will show the work to be covered by the Division of Structures plans, including a specific location for the pumping plant and storage box, the average and maximum pumping rates and the power required.

839.4 Trash and Debris Considerations

Storm drain systems leading to pumping plants are to be designed to limit the inflow of trash and debris, as these may cause damage to the pump impellers and create a maintenance removal nuisance. Standard grate designs are effective at ensuring that trash and debris are screened out of the inflow, but where side opening or curb opening inlets are constructed, trash racks must be added to the inlet design. The only Standard Plan detail for curb opening designs is shown on Standard Plan D74B and is used in conjunction with Type GDO inlets. On those occasions where pipe risers with side opening inlets are part of the system, refer to Standard Plan D93C for appropriate trash rack design details.

839.5 Maintenance Consideration

Access to the pumping plant location for both maintenance personnel and maintenance vehicles is generally provided by way of paved access road or city street. One parking space minimum is to be provided in the vicinity of the pumping plant. An area light is generally provided when it is determined that neither the highway lighting nor the street lighting is adequate. Access to the pumping plant for maintenance from the top of the cut slope generally consists of a stairway located adjacent to the pumping plant. The stairway generally extends from the top of cut slope to the toe of cut slope. Access to the pump control room should be through a vertical doorway with the bottom above flood level, and never through a hatch.

839.6 Groundwater Considerations

As the lowest point in the storm drain system, pumping plants are particularly susceptible to problems associated with rises in groundwater tables. Where the foundation of pump houses or associated storage boxes are at an elevation where they would be subjected to existing or future groundwater tables, sealing around the base of the foundation is necessary. The use of bentonite or other impervious material is typically sufficient in keeping groundwater from welling up through the relatively pervious structure backfill.

Sealing requirements will typically be specified by the Division of Structures during the pump plant design. However, the district should provide any information relative to historical groundwater levels or fluctuations which would be of importance, or known plans by local or regional water districts to modify recharge patterns in a manner that could impact the design.

CHAPTER 850 PHYSICAL STANDARDS

Topic 851 - General

Index 851.1 - Introduction

This chapter deals with the selection of drainage facility material type and sizes including pipes, pipe liners, pipe linings, drainage inlets and trench drains.

851.2 Selection of Material and Type

The choice of drainage facility material type and size is based on the following factors:

(1) *Physical and Structural Factors.* Of the many physical and structural considerations, some of the most important are:

(a) Durability.

(b) Headroom.

(c) Earth Loads.

(d) Bedding Conditions.

(e) Conduit Rigidity.

(f) Impact.

(g) Leak Resistance.

(2) *Hydraulic Factors.* Hydraulic considerations involve:

(a) Design Discharge.

(b) Shape, slope and cross sectional area of channel.

(c) Velocity of approach.

(d) Outlet velocity.

(e) Total available head.

(f) Bedload.

(g) Inlet and outlet conditions.

(h) Slope.

(i) Smoothness of conduit.

(j) Length.

Suggested values for Manning's Roughness coefficient (n) for design purposes are given in Table 851.2 for each type of conduit. See Index 866.3 for use of Manning's formula.

Topic 852 - Pipe Materials

852.1 Reinforced Concrete Pipe (RCP)

(1) *Durability.* RCP is generally precast prior to delivery to the project site. The durability of reinforced concrete pipe can be affected by abrasive flows or acids, chlorides and sulfate in the soil and water. See Index 855.2 Abrasion, and Index 855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates.

The following measures increase the durability of reinforced concrete culverts:

(a) Cover Over Reinforcing Steel. Additional cover over the reinforcing steel should be specified where abrasion is likely to be severe as to appreciably shorten the design service life of a concrete culvert. This extra cover is also warranted under exposure to corrosive environments, see Index 855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates. Extra cover over the reinforcing steel does not necessarily require extra wall thickness, as it may be possible to provide the additional cover and still obtain the specified D-load with standard wall thicknesses.

(b) Increase cement content.

(c) Reduce water content.

(d) Invert paving/plating.

(2) *Indirect Design Strength Requirements.*

(a) Design Standards. The “D” load strength of reinforced concrete pipe is determined by the load to produce a 0.01 inch crack under the “3-edge bearing test” called for in AASHTO Designations M 170, M 207M/M 207, and M 206M/M 206 for circular reinforced pipe, oval shaped reinforced pipe, and reinforced concrete pipe arches, respectively.

(b) Height of Fill. See Topic 856.

Table 851.2

Manning "n" Value for Alternative Pipe Materials[(1)]

Type of Conduit		Recommended Design Value	"n" Value Range
Corrugated Metal Pipe [(2)]			
(Annular and Helical) [(3)]			
2⅔" x ½"	corrugation	0.025	0.022 - 0.027
3" x 1"	"	0.028	0.027 - 0.028
5" x 1"	"	0.026	0.025 - 0.026
6" x 2"	"	0.035	0.033 - 0.035
9" x 2½"	"	0.035	0.033 - 0.037
Concrete Pipe			
Pre-cast		0.012	0.011 - 0.017
Cast-in-place		0.013	0.012 - 0.017
Concrete Box		0.013	0.012 - 0.018
Plastic Pipe (HDPE and PVC)			
Smooth Interior		0.012	0.010 - 0.013
Corrugated Interior		0.022	0.020 - 0.025
Spiral Rib Metal Pipe			
¾" (W) x 1" (D) @ 11½" o/c		0.013	0.011 - 0.015
¾" (W) x ¾" (D) @ 7½" o/c		0.013	0.012 - 0.015
¾" (W) x 1" (D) @ 8½" o/c		0.013	0.012 - 0.015
Composite Steel Spiral Rib Pipe		0.012	0.011 - 0.015
Steel Pipe, Ungalvanized		0.015	--
Cast Iron Pipe		0.015	--
Clay Sewer Pipe		0.013	--
Polymer Concrete Grated Line Drain		0.011	0.010 - 0.013

Notes:

(1) Tabulated n-values apply to circular pipes flowing full except for the grated line drain. See Note 5.

(2) For lined corrugated metal pipe, a composite roughness coefficient may be computed using the procedures outlined in the HDS No. 5, Hydraulic Design of Highway Culverts.

(3) Lower n-values may be possible for helical pipe under specific flow conditions (refer to FHWA's publication Hydraulic Flow Resistance Factors for Corrugated Metal Conduits), but in general, it is recommended that the tabulated n-value be used for both annular and helical corrugated pipes.

(4) For culverts operating under inlet control, barrel roughness does not impact the headwater. For culverts operating under outlet control barrel roughness is a significant factor. See Index 825.2 Culvert Flow.

(5) Grated Line Drain details are shown in Standard Plan D98C and described under Index 837.2(6) Grated Line Drains. This type of inlet can be used as an alternative at the locations described under Index 837.2(5) Slotted Drains. The carrying capacity is less than 18-inch slotted (pipe) drains.

(3) *Shapes.* Reinforced concrete culverts are available in circular and oval shapes. Reinforced Concrete Pipe Arch (RCPA) shapes have been discontinued by West Coast manufacturers.

In general, the circular shaped is the most economical for the same cross-sectional area. Oval shapes are appropriate for areas with limited head or overfill or where these shapes are more appropriate for site conditions. A convenient reference of commercially available products and shapes is the AASHTO publication, "A Guide to Standardized Highway Drainage Products".

(4) *Non-Reinforced Concrete Pipe Option.* Non-reinforced concrete pipe may be substituted at the contractor's option for reinforced concrete pipe for all sizes 36 inches in diameter and smaller as long as it conforms to Section 65 of the Standard Specifications. Non-Reinforced concrete pipe is not affected by chlorides or stray currents and may be used in lieu of RCP in these environments without coating or the need to provide extra cover over reinforcement.

(5) *Direct Design Method - RCP.* (Contact DES - Structures Design)

852.2 Concrete Box and Arch Culverts

(1) *Box Culverts.* Single and multiple span reinforced concrete box culverts are completely detailed in the Standard Plans. For cast-in-place construction, strength classifications are shown for 10 feet and 20 feet overfills. Precast reinforced concrete box culverts require a minimum of 1 foot of overfill and are not to exceed 12 feet in span length. Special details are necessary if precast boxes are proposed as extensions for existing box culverts. Where the use of precast box culverts is applicable, the project plans should include them as an alternative to cast-in-place construction. Because the standard measurement and payment clauses for precast RCB's differ from cast-in-place construction, precast units must be identified as an alternative and the special provision must be appropriately modified.

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

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

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

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

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

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

(1) *Hydraulics.* Annular and helical corrugated steel pipe configurations are applicable in the situations where velocity reduction is important or if a culvert is being designed with an inlet control condition. Spiral rib pipe, on the other

hand, may be more appropriate for use in stormdrain situations or if a culvert is being designed with an outlet control condition. Spiral rib pipe has a lower roughness coefficient (Manning's “n”) than other corrugated metal pipe profiles.

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

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

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

- Protective Coatings - The necessity for any coating should be determined considering hydraulic conditions, local experience, possible environmental impacts, and long-term economy. Approved protective coatings are bituminous asphalt, asphalt mastic and polymeric sheet, which can be applied to the inside and/or outside of the pipe; and polyethylene for composite steel spiral ribbed pipe which is a steel spiral ribbed pipe externally pre-coated with a polymeric sheet, and internally polyethylene lined. All of these protective coatings are typically shop-applied prior to delivery to the construction site. Polymeric sheet coating provides much improved corrosion resistance over bituminous coatings and can be considered to typically allow achievement of a 50-year maintenance-free service life without need to increase thickness of the steel pipe. To ensure that a damaged coating does not lead to premature catastrophic failure, the base steel thickness for pipes that are to be coated with a polymeric sheet must be able to provide a minimum 10-year service life prior to application of the polymeric material. In addition, a bituminous lining or bituminous paving can be applied over a bituminous coating primer on the inside of the pipe for extra corrosion or abrasion protection (see Section 66 of the Standard Specifications).

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

  Where the materials report indicates that soil side corrosion is expected, a bituminous asphalt coating which is hot-dipped to cover the entire inside and outside of the pipe or an exterior application of polymeric sheet, as provided in the Standard Specifications, combined with galvanizing of steel, is usually effective in forestalling accelerated corrosion on the backfill side of the pipe. Where soil side corrosion is the only, or primary, factor leading to deterioration, the bituminous asphalt protection layer described above is typically expected to add up to 25 years of service life to an uncoated (i.e., plain galvanized) pipe. A polymeric sheet coating is typically expected to provide up to 50-years of service life to an uncoated pipe. For locations where water side corrosion and/or abrasion is of concern, protective coatings, or protective coatings with pavings, or protective coatings with linings, in combination with galvanizing will add to the culvert service life to a variable degree, depending upon site conditions and type of coating selected. Refer to Index 855.2 Abrasion, and Index 855.3 Corrosion. If hydraulic conditions at the culvert site require a lining on the inside of the pipe or a coating different than that indicated in the Standard Specifications, then the different requirements must be described in the Special Provisions.

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

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

(b) Aluminized Steel (Type 2). Evaluations of aluminized steel (type 2) pipe in place for over 40 years have provided data that substantiate a design service life with respect to corrosion resistance equivalent to aluminum pipe. Therefore, for pH values between 5.5 and 8.5, and minimum resistivity values in excess of 1500 ohm-cm, 0.064 inch aluminized steel (type 2) is considered to provide a 50 year design service life. Where abrasion is of concern, aluminized steel (type 2) is considered to be roughly equivalent to galvanized steel. Bituminous coatings are not recommended for corrosion protection, but may be used in accordance with Table 855.2C for abrasion resistance. A concrete invert may also be considered where abrasion is of concern. For pH ranges outside the 5.5 and 8.5 limits or minimum resistivity values below 1500 ohm-cm, aluminized steel (type 2) should not be used. In no case should the thickness of aluminized steel (type 2) be less than the minimum structural requirements for a given diameter of galvanized steel. Refer to Index 855.2 Abrasion, and Index 855.3 Corrosion.

The AltPipe Computer Program is also available to help designers estimate service life for various corrosive/abrasive conditions. See http://www.dot.ca.gov/hq/oppd/altpipe.htm

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

(a) Design Standards.

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

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

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

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

(5) *Invert Protection.* Refer to Index 855.2 Abrasion. Invert protection should be considered for corrugated steel culverts exposed to excessive wear from abrasive flows or corrosive water. Severe abrasion usually occurs when the flow velocity exceeds 12 feet per second to 15 feet per second and contains an abrasive bedload of sufficient volume. When severe abrasion or corrosion is anticipated, special designs should be investigated and considered. Typical invert protection includes invert paving with portland cement concrete with wire mesh reinforcement, and invert lining with metal plate. The paving limits for invert linings are site specific and should be determined by field review. Additional metal thickness will increase service life. Reducing the velocity within the culvert is an effective method of preventing severe abrasion. Index 853.6 provides additional guidance on invert paving with concrete.

(6) *Spiral Rib Steel.* Galvanized steel spiral rib pipe is fabricated using sheet steel and continuous helical lock seam fabrication as used for helical corrugated metal pipe. The manufacturing complies with Section 66, "Corrugated Metal Pipe," of the Standard Specifications, except for profile and fabrication requirements. Spiral rib pipe is fabricated with either: three rectangular ribs spaced midway between seams with ribs 3/4" wide x 3/4" high at a maximum rib pitch of 7-1/2 inches, two rectangular ribs and one half-circle rib equally spaced between seams with ribs 3/4" wide x 1" high at a maximum rib pitch of 11-1/2 inches with the half-circle rib diameter spaced midway between the rectangular ribs, or two rectangular ribs equally spaced between seams with ribs 3/4" wide x 1" high at a maximum rib pitch of 8-1/2 inches.

Aluminized steel spiral rib pipe, type 2 (ASSRP) is available in the same sizes as galvanized steel spiral rib and will support the same fill heights (the aluminizing is simply a replacement coating for zinc galvanizing that allows thinner steel to be placed in certain corrosive environments. See Figure 855.3A for the acceptable pH and resistivity ranges for placement of aluminized steel pipes). Tables 856.3E, F & G give the maximum height of overfill for steel spiral rib pipe constructed under the acceptable methods contained in the Standard Specifications and essentials discussed in Index 829.2.

852.4 Corrugated Aluminum Pipe, Aluminum Spiral Rib Pipe and Pipe Arches

Corrugated aluminum pipe, aluminum spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. For larger diameters, arch spans or special shapes see Index 852.6. Corrugated aluminum pipe and pipe arches are available in various corrugation profiles with helical and annular corrugations. Helical corrugated pipe must be specified if anticipated heights of cover exceed the tabulated values for annular corrugated pipe. Non-standard pipe diameters and arch sizes are also available. Aluminum spiral rib pipe is similar to spiral rib steel and is available in several helical corrugation patterns.

(1) *Hydraulics*. Corrugated aluminum pipe comes in various corrugated profiles. Annular and helical corrugated aluminum pipe configurations are applicable in the situations where velocity reduction is important or if a culvert is being designed with an inlet control condition. Spiral rib pipe, on the other hand, may be more appropriate for use in stormdrain situations or if a culvert is being designed with an outlet control condition. Spiral rib pipe has a

lower roughness coefficient (Manning's "n") than other corrugated metal pipe profiles.

(2) *Durability*. Aluminum culverts or stormdrains may be specified as an alternate culvert material. When a 50-year maintenance-free service life of aluminum pipe is required the pH and minimum resistivity, as determined by California Test Method 643, must be known and the following conditions met:

(a) The pH of the soil, backfill, and effluent is within the range of 5.5 and 8.5, inclusive. Bituminous coatings are not recommended for corrosion protection or abrasion resistance. However, a concrete invert lining may be considered. Abrasive potential must be estimated from bed material that is present and anticipated flow velocities. Refer to Index 855.1 for a discussion of maintenance-free service life and Index 855.2 Abrasion, and Index 855.3 Corrosion prior to selecting aluminum as an allowable alternate.

(b) The minimum resistivity of the soil, backfill, and effluent is 1500 ohm-cm or greater.

(c) Aluminum culverts should not be installed in an environment where other aluminum culverts have exhibited significant distress, such as extensive perforation or loss of invert, for whatever reason, apparent or not.

(d) Aluminum may be considered for side drains in environments having the following parameters:

- When pH is between 5.5 and 8.5 and the minimum resistivity is between 500 and 1500 ohm-cm.
- When pH is between 5.0 and 5.5 or between 8.5 and 9.0 and the minimum resistivity is greater than 1500 ohm-cm.

For these conditions, the Corrosion Technology Branch in METS should be contacted to confirm the advisability of using aluminum on specific projects.

(e) Aluminum must not be used as a section or extension of a culvert containing steel sections.

(3) *Strength Requirements.* The strength requirements for corrugated aluminum pipe and pipe arches fabricated under the acceptable methods contained in the Standard Specifications, are given in Tables 856.3H, I & J. See Table 856.3K and Table 856.3L for aluminum spiral rib pipe. Tables 856.3H through L are based on the material properties of H-32 temper aluminum. Additional cover heights can be achieved for an aluminum section when H-34 temper material is used. Contact DES-Structures Design for a special design using H-34 temper material.

(a) Design Standards.

- Corrugation Profiles - Corrugated aluminum pipe and pipe arches are available in 2⅔" x ½" and 5" x 1" profiles with helical or annular corrugations. Aluminum spiral rib pipe is available in a ¾" x ¾" x 7½" or a ¾" x 1" x 11½" helical corrugation profile.
- Metal thickness - Corrugated aluminum pipe and pipe arches are available in the thickness as indicated on Tables 856.3H, I & J. Where a maximum overfill is not listed on these tables, the pipe or pipe arch is not normally available in that thickness. All pipe sections provided in Table 856.3 meet handling and installation flexibility requirements of AASHTO LRFD. Aluminum spiral rib pipe are available in the thickness as indicated on Tables 856.3K & L.
- Height of Fill - The allowable overfill heights for corrugated aluminum pipe and pipe arches for various diameters and metal thicknesses are shown on Tables 856.3H, I & J. For aluminum spiral rib pipe, overfill heights are shown on Tables 856.3K, & L.

(4) *Shapes*. Corrugated aluminum pipe, aluminum spiral rib pipe and pipe arches are available in the diameters and arch shapes as indicated on the maximum height of cover tables. Helical corrugated pipe must be specified if anticipated

heights of cover exceed the tabulated values for annular corrugated pipe.

For larger diameters, arch spans or special shapes, see Index 852.5. Non-standard pipe diameters and arch sizes are also available.

(5) *Invert Protection.* Invert protection of corrugated aluminum is not recommended.

(6) *Spiral Rib Aluminum.* Aluminum spiral rib pipe is fabricated using sheet aluminum and continuous helical lock seam fabrication as used for helical corrugated metal pipe. The manufacturing complies with Section 66, "Corrugated Metal Pipe," of the Standard Specifications, except for profile and fabrication requirements. Aluminum spiral rib pipe is fabricated with either: three rectangular ribs spaced midway between seams with ribs 3/4" wide x 3/4" high at a maximum rib pitch of 7-1/2 inches or two rectangular ribs and one half-circle rib equally spaced between seams with ribs 3/4" wide x 1" high at a maximum rib pitch of 11-1/2 inches with the half-circle rib diameter spaced midway between the rectangular ribs. Figure 855.3A should be used to determine the limitations on the use of spiral rib aluminum pipe for the various levels of pH and minimum resistivity.

852.5 Structural Metal Plate

(1) *Pipe and Arches.* Structural plate pipes and arches are available in steel and aluminum for the diameters and thickness as shown on Tables 856.3M, N, O & P.

(2) *Strength Requirements.*

(a) Design Standards.

- Corrugation Profiles - Structural plate pipe and arches are available in a 6" x 2" corrugation for steel and a 9" x 2½" corrugation profile for aluminum.
- Metal Thickness - structural plate pipe and pipe arches are available in thickness as indicated on Tables 856.3M, N, O & P.
- Height of Fill - The allowable height of cover over structural plate pipe and pipe arches for the available diameters and thickness are shown on Tables 856.3M, N, O & P.

Where a maximum overfill is not listed on these tables, the pipe or arch size is not normally available in that thickness. All pipe sections provided in Table 856.3 conform to handling and installation flexibility requirements of AASHTO LRFD. Strutting of culverts, as depicted on Standard Plan D88A, is typically necessary if the pipe is used as a vertical shaft or if the backfill around the pipe is being removed in an unbalanced manner.

(b) Basic Premise. To properly use the above mentioned tables, the designer should be aware of the premises on which the tables are based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications, the conditions of cover, and pipe or arch size required by the plans and the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.

(c) Limitations. In using the tables, the following restrictions should be kept in mind.

- The values given for each size of structural plate pipe or arch constitute the maximum height of overfill or cover over the pipe or arch for the thickness of metal and kind of corrugation.
- The thickness shown is the structural minimum. For steel pipe or pipe arches, where abrasive conditions are anticipated, additional metal thickness for the invert plate(s) or a paved invert should be provided when required to fulfill the design service life requirements. Table 855.2C may be used. See Index 855.2 Abrasion and Tables 855.2A, 855.2D and 855.2F.
- Where needed, adequate provisions for corrosion resistance must be made to

achieve the required design service life called for in the references mentioned herein.

- Tables 856.3M & P show the limit of heights of cover for structural plate arches based on the supporting soil sustaining a bearing pressure of 3 tons per square foot at the corners.

(d) Special Designs. If the height of overfill exceeds the tabular values, or if the foundation investigation reveals that the supporting soil will not develop the bearing pressure on which the overfill heights for structural plate pipe or pipe arches are based, a special design prepared by DES - Structures Design is required.

(3) Arches. Design details with maximum allowable overfills for structural plate arches, with cast in place concrete footings may be obtained from DES - Structures Design.

(4) Vehicular Underpasses. Design details with maximum allowable overfills for structural plate vehicular underpasses with spans from 12 feet 2 inches to 20 feet 4 inches, inclusive, are given in the Standard Plans. These designs are based on "factored" bearing soil pressures from 2.5 tons per square foot to 11 tons per square foot.

(5) Special Shapes.

(a) Long Span.

- Arch
- Low Profile Arch
- High Profile Arch

(b) Ellipse. (Text Later)

- Vertical
- Horizontal

(6) Tunnel Liner Plate. The primary applications for tunnel liner plate include lining large structures in need of a structural repair, or culvert installations through an existing embankment that can be constructed by conventional tunnel methods. Typically, tunnel liner plate is not used for direct burial applications where structural metal plate pipe is recommended. DES - Structures Design will prepare designs upon request. See Index 853.7 for structural repairs.

852.6 Plastic Pipe

Plastic pipe is a generic term which currently includes two independent materials; the Standard Specifications states plastic pipe shall be made of either high density polyethylene (HDPE) or polyvinyl chloride (PVC) material. See Index 852.6(2)(a) Strength Requirements for allowed materials and wall profile types.

(1) Durability. Caltrans standards regarding the durability of plastic pipe are based on the long term performance of its material properties. Both forms of plastic pipe culverts (HDPE and PVC) exhibit good abrasion resistance and are virtually corrosion free. See Index 855.2 Abrasion and Index 855.5 Material Susceptibility to Fire. Also, see Tables 855.2A, 855.2E and 855.2F. The primary environmental factor currently considered in limiting service life of plastic materials is ultraviolet (UV) radiation, typically from sunlight exposure. While virtually all plastic pipes contain some amount of UV protection, the level of protection is not equal. Polyvinyl chloride resins used for pipe rarely incorporate UV protection (typically Titanium Dioxide) in amounts adequate to offset long term exposure to direct sunlight. Therefore, frequent exposure (e.g., cross culverts with exposed ends) can lead to brittleness and such situations should be avoided. Conversely, testing performed to date on HDPE products conforming to specification requirements for inclusion of carbon black have exhibited adequate UV resistance. PVC pipe exposed to freezing conditions can also experience brittleness and such situations should be avoided if there is potential for impact loadings, such as maintenance equipment or heavy (3" or larger) bedload during periods of freeze. Plastic pipes can also fail from long term stress that leads to crack growth and from chemical degradation. Improvements in plastic resin specifications and testing requirements has led to increased resistance to slow crack growth. Inclusion of anti-oxidants in the material formulation is the most common form of delaying the onset of chemical degradation, but

more thorough testing and assessment protocols need to be developed to more accurately estimate long term performance characteristics and durability.

(2) *Strength Requirements.*

(a) Design Standards

- Materials - Plastic pipe shall be either Type C (corrugated exterior and interior) corrugated polyethylene pipe, Type S (corrugated exterior and smooth interior) corrugated polyethylene pipe, or corrugated polyvinyl chloride pipe.
- Height of Fill - The allowable overfill heights for plastic pipe for various diameters are shown in Tables 856.4 and 856.5.

852.7 Special Purpose Types

(1) *Smooth Steel.* Smooth steel (welded) pipe can be utilized for drainage facilities under conditions where corrugated metal or concrete pipe will not meet the structural or design service life requirements, or for certain jacked pipe operations (e.g., auger boring).

(2) *Composite Steel Spiral Rib Pipe.* Composite steel spiral rib pipe is a smooth interior pipe with efficient hydraulic characteristics. See Table 851.2.

Composite steel spiral rib pipe with its interior polyethylene liner exhibits good abrasion resistance and also resists waterside corrosion found in a typical stormdrain or culvert environment. The exterior of the pipe is protected with a polyethylene film, which offers resistance to corrosive backfills. The pipe will meet a 50-year maintenance-free service life under most conditions. See Table 856.3G for allowable height of cover.

(3) *Proprietary Pipe.* See Index 110.10 for further discussion and guidelines on the use of proprietary items.

Topic 853 - Pipe Liners and Linings for Culvert Rehabilitation

853.1 General

This topic discusses alternative pipe liner and pipe lining materials specifically intended for culvert repair and does not include materials used for Trenchless Excavation Construction (e.g., pipe jacking, pipe ramming, augur boring), joint repair, various types of grouting, or standard pipe materials that are presented elsewhere in Chapter 850 and in the Standard Plans and Standard Specifications.

Many new products and techniques have been developed that often make complete replacement with open cut as shown in the Standard Plans unnecessary. When used appropriately, these new products and techniques can benefit the Department in terms of increased mobility, cost, and safety to both the public and contractors. Design Information Bulletin 83 (DIB 83) outlines a collection of procedures that are cost-effective for their location and that will meet the needs of their particular area, supplementing Topic 853. Use the following link: http://www.dot.ca.gov/hq/oppd/dib/dibprg.htm for further information.

853.2 Caltrans Host Pipe Structural Philosophy

In general, if the host (i.e., existing) pipe cannot be made capable of sustaining design loads, it should be replaced rather than rehabilitated. This is a conservative approach and when followed eliminates the need to make a detailed evaluation of the liner's ability to effectively accept and support dead and live loads. Prior to making the decision whether or not to rehabilitate the culvert and/or which method to choose, a determination of the structural integrity of the host pipe must be made. If rehabilitation of the culvert is determined to be a feasible option, existing voids within the culvert backfill or in the base material under the existing culvert identified either by Maintenance (typically as part of their culvert management system) or already noted in the Geotechnical Design Report, should be filled with grout to re-establish its load carrying capability. Therefore, structural considerations for pipe liners are generally limited to their ability to withstand construction handling and/or grouting pressures. When a structural repair is needed, contact Underground Structures within DES – Structures Design. See Index 853.7.

853.3 Problem Identification and Coordination

Before various alternatives for liners or linings can be selected, the first step following a site investigation which may include taking soil and water samples and pipe wall thickness measurements, is to determine the actual cause of the problem. Relative to Caltrans host pipe structural philosophy, the host pipe may be in need of stabilization, rehabilitation or replacement. Further, it will need to be determined if the structure is at the end of its maintenance-free service life, whether it has been damaged by mechanical abrasion, or corrosion (or both) and if there are any changes to the hydrology or habitat (e.g. fish passage). To make these determinations, the Project Engineer should coordinate with the District Maintenance Culvert Inspection team, Hydraulics and Environmental units. Further assistance may be needed from Geotechnical Design, the Corrosion Technology Branch within DES, Underground Structures and/or Structures Maintenance within DES. Prior to a comprehensive inspection either by trained personnel or camera, it may also be necessary to first clean out the culvert. Problem identification and assessment, and coordination with Headquarters and DES, is discussed in greater detail in DIB 83. Use the following link; http://www.dot.ca.gov/hq/oppd/dib/dib83-01-7.htm#7-1-6

853.4 Alternative Pipe Liner Materials

Similar to the basic policy in Topic 857.1 for alternative pipes, when two or more liner materials meet the design service life and minimum thickness requirements for various materials that are outlined under Topic 855, as well as hydraulic requirements, the plans and specifications should provide for alternative pipe liners to allow for optional selection by the contractor. A table of allowable alternative pipe liner materials for culverts and drainage systems is included as Table 853.1A. This table also identifies the various diameter range limitations and whether annular space grouting is needed. Sliplining consists of sliding a new culvert inside an existing distressed culvert as an alternative to total replacement. See DIB No 83; http://www.dot.ca.gov/hq/oppd/dib/dib83-01-6.htm#6-1-3-1.

The plastic pipeliners listed in the notes under Table 853.1A are installed as slipliners, however, other standard pipe types that are described in Topic 852 (e.g., metal), may be equally viable as material options to be added as sliplining alternatives.

Table 853.1A Allowable Alternative Pipe Liner Materials

Allowable Alternatives	Diameter Range [(1)]	Annular Space Grouting
PP [(2)]	15" – 120"	Yes
CIPP	8" – 96"	No
MSWPVCPLED	6" – 30"	No
SWPVCPLFD	21" – 108"	Yes

Abbreviations:

PP	–	Plastic Pipe (sliplining)
CIPP	–	Cured in Place Pipe
SWPVCPLFD	–	Spiral Wound PVC Pipe Liner (Fixed Diameter)
MSWPVCPLED	–	Machine Spiral Wound PVC Pipe Liner (Expandable Diameter)

Note:

(1) Headquarters approval needed for pipe liner diameters 60 inches or larger. Diameter range represents liners only, not Caltrans standard pipe.

(2) The designer must edit the following plastic pipeliner list within SSP 15-6.10 to suit the work:

- Type S corrugated high density polyethylene (HDPE) pipe conforming to the provisions in Section 64, "Plastic Pipe," of the Standard Specifications; or
- Standard Dimension Ratio (SDR) 35 polyvinyl chloride (PVC) pipe conforming to the requirements in AASHTO Designation: M 278 and ASTM Designation: F 679; or
- Polyvinyl chloride (PVC) closed profile wall pipe conforming to the requirements in ASTM Designation: F 1803, F 794 (Series 46); or
- Polyvinyl chloride (PVC) dual wall corrugated pipe conforming to the requirements in ASTM Designation: F 794 (Series 46), and ASTM Designation F 949; or
- High density polyethylene (HDPE) solid wall pipe conforming to the requirements in AASHTO M 326 and ASTM Designation: F 714; or

- Large diameter high density polyethylene (HDPE) closed profile wall pipe conforming to the requirements in ASTM Designation: F 894.

Table 853.1B provides a guide for plastic pipeliner selection in abrasive conditions to achieve a 50-year maintenance-free service life.

For further information on sliplining using plastic pipe liners including available dimensions and stiffness, see DIB 83. Use the following link: http://www.dot.ca.gov/hq/oppd/dib/dib83-01-6.htm#6-1-3-1-1

853.5 Cementitious Pipe Lining

This method may be used to line corroded corrugated steel pipes ranging from 12 inches to a maximum of 36 inches diameter and involves lining an existing culvert with concrete, shotcrete or mortar using a lining machine. If the bedload is abrasive, alternative cementitious materials such as calcium aluminate mortar or geopolymer mortar may be selected from the Authorized Materials list for cementitious pipeliners. See Table 855.2F and Section 15-6.14 of the Standard Specifications for specifications. Regardless of type of cementitious material used, the resulting lining is a minimum of one inch thick when measured over the top of corrugation crests and has a smooth surface texture. As with other liners, the pipes must first be thoroughly cleaned and dried. For diameters between 12 and 24 inches, the cement mortar is applied by robot. The mortar is pumped to a head, which rotates at high speed using centrifugal force to place the mortar on the walls. A conical-shaped trowel attached to the end of the machine is used to smooth the walls. The maximum recommended length of small-diameter pipe that can be lined using this method is approximately 650 feet. Although this method will line larger diameter pipes, it is mostly appropriate for non-human entry pipes (less than 30 inches). Generally, most problems with steel pipe are limited to the lower 180 degrees, therefore, in larger diameter metal pipes where human entry is possible, invert paving may be all that is required. See Index 853.6.

853.6 Invert Paving with Concrete

(1) *Existing Corrugated Metal Pipe (CMP).* One of the most effective ways to rehabilitate corroded and severely deteriorated inverts of CMP that are large enough for human entry (with equipment) is by paving them with reinforced concrete shotcrete or authorized cementitious material. Standard Specification Section 15-6.04 includes specifications for preparing the surface of the culvert invert, installing bar reinforcement and anchorage devices, and paving the invert with concrete, shotcrete or authorized cementitious material. For most non-abrasive sites, concrete may comply with the requirements for minor concrete or shotcrete. See index 110.12 Tunnel Safety Orders. Generally, this method is feasible for pipes 48 inches in diameter and larger. If abrasion is present, see Table 855.2F for minimum material thickness of concrete or authorized material. Concrete should have a minimum compressive strength of 6,000 psi at 28 days and the aggregate source should be harder material than the streambed load and have a high durability index (consult with District Materials Branch for sampling and recommendation). The maximum grading specified (1.5 inch) for coarse aggregate may need to be modified if the concrete must be pumped. The abrasion resistance of cementitious materials is affected by both its compressive strength and hardness of the aggregate. There is a correlation between decreasing the water/cement ratio, increasing compressive strength and increasing abrasion resistance. Therefore, where abrasion is a significant factor, the lowest practicable water/cement ratios and the hardest available aggregates should be used.

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

Table 853.1B

Guide for Plastic Pipeliner Selection in Abrasive Conditions(2) to Achieve 50 Years of Maintenance-Free Service Life

		Abrasion Level(1)		
MATERIAL		4	5	6
Type S corrugated polyethylene pipe		-	-	-
Standard Dimension Ratio (SDR) 35 PVC (3)	(46 psi)	4" – 48"	12"- 48"	36"– 48"
	(75 psi)	18" – 48"	18" – 48"	30" – 48"
	(115 psi)	18" – 48"	18" – 48"	27" – 48"
PVC closed profile wall (ASTM F 1803)		18" – 60"	42"– 60"	-
Corrugated PVC (ASTM F 794 & F 949)	(46 psi)	18" – 36"	-	-
	(115 psi)	15"	-	-
Standard Dimension Ratio (SDR) HDPE(3) conforming to: AASHTO M 326 and ASTM Designation F 714	SDR 41	10" – 63"	36" – 63"	-
	SDR 32.5	8" – 63"	30" – 63"	-
	SDR 26	6" – 63"	24" – 63"	-
	SDR 21	5" – 63"	20" – 63"	54" – 63"
	SDR 17	5" – 55"	16" – 55"	42" – 55"
	SDR 15.5	5" – 48"	14" – 48"	42" – 48"
	SDR 13.5	5" – 42"	12" – 42"	34" – 42"
	SDR 11	5" – 36"	10" – 36"	28" - 36"
	SDR 9	5" – 24"	8" – 24"	22"
Polyethylene (PE) large diameter profile wall sewer and drain pipe as specified in ASTM F 894	RSC 160	18" – 120"	120"	-
Note: RSC = Ring Stiffness Class	RSC 250	33" – 108"	96" – 108"	-

Notes:

(1) See Tables 855.2A and 855.2F for Abrasion Level Descriptions and minimum thickness.

(2) No restrictions for Abrasion Levels 1 through 3.

(3) Pipe designated SDR is measured to outside diameter.

Consult the District Hydraulic Branch for a recommendation.

Where there is significant loss of the pipe invert, it may be necessary to tie the concrete to more structurally sound portions of the pipe wall in order to transfer compressive thrust of culvert walls into the invert slab to create a "mechanical" connection using welding studs, angle iron or by other means. When a mechanical connection is used, paving limits may vary up to 180 degrees for the internal angle. These types of repairs should be treated as a special design and consultation with the Headquarters Office of Highway Drainage Design within the Division of Design and the Underground Structures unit of Structures Design within the Division of Engineering Services (DES) is advised. Depending on the size of the culvert being paved, pipes with significant invert loss often also have a significant loss of structural backfill with voids present. Where large voids are present, consultation with Geotechnical Services within the Division of Engineering Services (DES) is advised to develop a grouting plan.

See DIB 83 for some invert paving case studies using the following link: http://www.dot.ca.gov/hq/oppd/dib/dib83-01-12.htm#h

(2) *Existing RCB and RCP*. For existing reinforced concrete boxes (RCB) and reinforced concrete pipes (RCP) with worn inverts and exposed reinforcing steel (generally from abrasive bedloads), the same paving thickness considerations outlined under Index 853.6(1) will apply. However, depending on the structural condition, the existing steel reinforcement may need to be augmented. Consultation with Structures Maintenance and Underground Structures within DES is recommended.

(3) *Existing Plastic Pipe.* Generally, concrete invert paving is not feasible for plastic pipes because the cement will not adhere to plastic. However, it may be possible to create a "mechanical" connection by other means but these types of repairs should be treated as a special design and consultation with the Headquarters Office of Highway Drainage Design within the Division of Design and the Underground Structures unit of Structures Design within the Division of Engineering Services (DES) is advised.

853.7 Structural Repairs with Steel Tunnel Liner Plate

Cracks in RCP greater than 0.1 inch in width and flexible metal pipes with deflections beyond 10 – 12 percent may indicate a serious condition. When replacement is not an option for existing human entry pipes in need of structural repair, an inspection by Structures Maintenance and a structural analysis by Underground Structures within DES are recommended. Further assistance may be needed from Geotechnical Design and/or the Corrosion Unit within DES.

Two flange or four flange steel tunnel liner plate can be specially designed by Underground Structures within DES as a structural repair to accommodate all live and dead loads. The flange plate lap joints facilitate internal bolt connections (structural metal plate requires access to both sides). After the rings have been installed, the annular space between the liner plates and the host pipe is grouted.

Topic 854 - Pipe Connections

854.1 Basic Policy

The Standard Specifications set forth general performance requirements for transverse field joints in all types of culvert and drainage pipe used for highway construction.

Table 857.2 indicates the alternative types of joints that are to be specified for different arch and circular pipe installations with regard to joint strength. The two joint strength types specified for culvert and drainage systems are identified as "standard" and "positive."

(1) *Joint Strength.* Joint strength is to be designated on the culvert list.

(a) Standard Joints. The "standard" joint is usually for pipes or arches not subject to large soil movement or disjointing forces. These "standard" joints are satisfactory for ordinarily installations, where tongue and groove or simple slip type joints are typically used. The "standard" joint type is generally adequate for underdrains.

(b) Positive Joints. "Positive" joints are for more adverse conditions such as the need to withstand soil movements or resist disjointing forces. Examples of these conditions are steep slopes, sharp curves, and poor foundation conditions. See Index 829.2 for additional discussion. "Positive" joints should always be designated on the culvert list for siphon installations.

(c) Downdrain Joints. Pipe "downdrain" joints are designed to withstand high velocity flows, and to prevent leaking and disjointing that could cause failure.

(d) Joint Strength Properties. A description of the specified joint strength properties tabulated in Section 61 "Culvert and Drainage Pipe Joints" of the Standard Specifications is as follows:

- Shear Strength. The shear strength required of the joint is expressed as a percentage of the calculated shear strength of the pipe at a transverse section remote from the joint. All joints, including any connections must be capable of transferring the required shear across the joint.

- Moment Strength. The moment strength required of the joint is expressed as a percent of the calculated moment capacity of the pipe on a transverse section remote from the joint.

- Tensile Strength. The tensile strength is that which resist the longitudinal force which tends to separate (disjoint) adjacent pipe sections.

- Joint Overlap.

  Integral Preformed Joint. The Joint overlap is the amount of protection of one culvert barrel into the adjacent culvert barrel by the amount specified for the size of pipe designated. The amount of required overlap will vary based on several factors (material type, diameter, etc.) and is designated on the Standard Plans and/or Standard Specifications.

  Any part of an installed joint that has less than ¼ inch overlap will be considered disjointed. Whenever the plans require that the culvert be constructed on a curve, specially manufactured sections of culvert will be required if the design joint cannot meet the minimum ¼ inch overlap requirement after the culvert section is placed on the specified curve.

  - Sleeve Joints. The joint overlap is the minimum sleeve width (typically defined by the width of a coupling band) required to engage both the culvert barrels which are abutted to each other.

(2) Joint Leakage. The ability of a pipe joint to prevent the passage of either soil particles or water defines its soiltightness or watertightness. These terms are relative and do not mean that a joint will be able to completely stop the movement of soil or water under all conditions. Any pipe joint that allows significant soil migration (piping) will ultimately cause damage to the embankment, the roadway, or the pipe itself. Therefore, site conditions, such as soil particle size, presence of groundwater, potential for pressure flow, etc., must be evaluated to determine the appropriate joint requirement. Other than solvent or fusion welded joints, almost all joints can exhibit some amount of leakage. Joint performance is typically defined by maximum allowable opening size in the joint itself or by the ability to pass a standardized pressure test. The following criteria should be used, with the allowable joint type(s) indicated on the project plans:

- Normal Joint. Many pipe joint systems are not defined as either soiltight or watertight. However, for the majority of applications, such as culverts or storm drains placed in well graded backfill and surrounding soils containing a minimum of fines; no potential for groundwater contact; limited internal pressure, hydraulic grade line below the pavement grade, etc., this type of joint is acceptable. All currently accepted joint types will meet or exceed "Normal Joint" requirements. The following non-gasketed joint types should not be used beyond the "Normal Joint" criteria range:

CMP	-Annular -Hat -Helical -Hugger -2-piece Integral Flange -Universal
PLASTIC	-Split Coupler -Bell/Spigot

- Soiltight Joint. This category includes those joints which would provide an enhanced level of security against leakage and soil migration over the normal joint. One definition of a soiltight joint is contained in Section 26.4.2.4(e) of the AASHTO Standard Specifications for Highway Bridges. In part, this specification requires that if the size of the opening through which soil might migrate exceeds 1/8 inch, the length of the channel (length of path along which the soil particle must travel, i.e., the coupling length) must exceed 4 times the size of the opening. Alternatively, AASHTO allows the joint to pass a hydrostatic test (subjected to approx. 4.6 feet of head) without leaking to be considered soiltight. Typical pipe joints that can meet this criteria are:

RCP and NRCP	-Flared Bell -Flushed Bell -Steel Joint-Flush Bell -Single or Double Offset Design (Flared or Flushed Bell) -Double Gasket -Tongue and Groove* -Self-Centering T & G*
CMP and SSRP	-Annular w/gasket -Hat w/gasket -Helical w/gasket -Hugger w/gasket -2-piece Int. Fl. w/gasket -Universal w/gasket
CSSRP	-Cuffed end w/gasket
PLASTIC	-Split Coupler w/gasket (premium) -Bell/Spigot w/gasket

* Where substantial differential settlement is anticipated, would only meet Normal Joint criteria.

Where soil migration is of concern, but leakage rate is not, a soiltight joint can be achieved in most situations by external wrapping of the joint area with filter fabric (see Index 831.4). Joints listed under both the normal joint and soiltight joint categories, with a filter fabric wrap, would be suitable in these conditions and would not require a gasket or sealant. In many cases, fabric wrapping can be less expensive than a rubber gasket or other joint sealant. Coordination with the District Materials Unit is advised to ensure that the class of filter fabric will withstand construction handling and screen fine soil particles from migrating through the joint.

- Watertight Joint. Watertight joints are specified when the potential for soil erosion or infiltration/exfiltration must be restricted, such as for downdrains, culverts in groundwater zones, etc. Watertight joint requirements are typically met by the use of rubber gasket materials as indicated in the Standard Specifications. The watertight certification test described in Standard Specification Section 61 requires that no leakage occur when a joint is tested for a period of 10 minutes while subjected to a head of 10 feet over the crown of the pipe. This is a test that is typically performed in a laboratory under optimal conditions not typical of those found in the field. Where an assurance of water tightness is needed, a field test should be specified. Designers should be aware that field tests can be relatively expensive, and should only be required if such assurance is critical. A field leakage rate in the range of 700 gallons to 1,000 gallons per inch of nominal diameter per mile of pipe length per day, with a hydrostatic head of 6 feet above the crown of the pipe, is not unusual for joints that pass the watertight certification test, and is sufficiently watertight for well graded, quality backfill conditions. Where conditions are more sensitive, a lower rate should be specified. Rates below 50 to 100 gallons per inch per mile per day are difficult to achieve and would rarely be necessary. For example, sanitary sewers are rarely required to have leakage rates below 200 gallons per inch per mile per day, even though they have stringent health and environmental restrictions. Field hydrostatic tests are typically conducted over a period of 24 hours or more to establish a valid leakage rate. Designers should also be aware that non-circular pipe shapes (CMP pipe arches, RCP oval shapes, etc.) should not be considered watertight even with the use of rubber gaskets or other sealants due to the lack of uniform compression around the periphery of the joint. Additionally, watertight joints specified for pressure pipe or siphon applications must meet the requirements indicated in Standard Specification Sections 65 and 66. Pipe joints that meet Standard Specification Section 61 water-tightness performance criteria are:

<u>RCP and NRCP</u>	-Flared Bell -Flushed Bell -Steel Joint-Flush Bell -Single or Double Offset Design (Flared or Flushed Bell) -Double Gasket
<u>CMP and SSRP</u>	-Hugger Bands (H-10, 12) w/gasket and double bolt bar -Annular Band w/gasket -Two Piece Integral Flange w/sleeve-type gasket*
<u>PLASTIC</u>	-Bell/Spigot w/gasket

* Acceptable as a watertight pipe only in downdrain applications and in 6, 8 and 10 inch diameters. Factory applied sleeve-type gaskets are to be used instead of O-ring or other sealants.

Table 854.1 provides information to help the designer select the proper joint under most conditions.

Topic 855 - Design Service Life

855.1 Basic Concepts

The prediction of design service life of drainage facilities is difficult because of the large number of variables, continuing changes in materials, wide range of environments, and use of various protective coatings. The design service life of a drainage facility is defined as the expected maintenance-free service period of each installation. After this period, it is anticipated major will be needed for the facility to perform as originally designed for further periods.

For all metal pipes and arches that are listed in Table 857.2, maintenance-free service period, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of perforation at any location on the culvert (See Figures 855.3A, 855.3B, and Tables 855.2D and 855.2F). AltPipe can be used to estimate service life of all circular metal pipe. See Index 857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe.

For reinforced concrete pipe (RCP), box (RCB) and arch (RCA) culverts, maintenance-free service period, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of exposed reinforcement at any point on the culvert. AltPipe can be used to estimate service life of reinforced concrete pipe (RCP), but not RCB, RCA or NRCP. See Index 857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe.

For non-reinforced concrete pipe culverts (NRCP), maintenance-free service period, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of perforation or major cracking with soil loss at any point on the culvert.

For plastic pipe, maintenance-free service period, with respect to corrosion, abrasion, and long term structural performance, is the number of years from installation until the deterioration reaches the point

Table 854.1
Joint Leakage Selection Criteria

JOINT TYPE ⇒ ⇓ SITE CONDITIONS	"NORMAL" JOINT	"SOIL TIGHT" JOINT	"WATER TIGHT" JOINT
SOIL FACTORS			
Limited potential for soil migration (e.g., gravel, medium to coarse sands, cohesive soil)	X	X	X
Moderate potential for soil migration (e.g., fine sands, silts)	X(1)	X	X
High potential for soil migration (e.g., very fine sands, silts of limited cohesion)		X(1)	X(1)
INFILTRATION / EXFILTRATION			
No concern over either infiltration or exfiltration.	X	X	X
Infiltration or exfiltration not permitted (e.g., potential to contaminate groundwater, contaminated plume could infiltrate)			X(2)
HYDROSTATIC POTENTIAL			
Installation will rarely flow full. No contact with groundwater.	X	X	X
Installation will occasionally flow full. Internal head no more than 10 feet over crown. No potential groundwater contact.		X	X
Installation may or may not flow full. Internal head no more than 10 feet over crown. May contact groundwater.			X
Possible hydrostatic head (internal or external) greater than 10 feet, but less than 25 ft(3).			X(2)

Notes:
"X" indicates that joint type is acceptable in this application. The designer should specify the most cost-effective option.
(1) Designer should specify filter fabric wrap at joint. See Index 831.4.
(2) Designer should consider specifying field watertightness test.
(3) Pipe subjected to hydrostatic heads greater than 25 ft should have joints designed specifically for pressure applications.

of perforation at any location on the culvert or until the pipe material has lost structural load carrying capacity typically represented by wall buckling or excessive deflection/deformation. AltPipe can be used to estimate service life of all plastic pipe. See Index 857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe. All types of culverts are subject to deterioration from corrosion, or abrasion, or material degradation.

Corrosion may result from active elements in the soil, water and/or atmosphere. Abrasion is a result of mechanical wear and depends upon the frequency, duration and velocity of flow, and the amount and character of bedload. Material degradation may result from material quality, UV exposure, or long term material structural performance.

To assure that the maintenance-free service period is achieved, alternative metal pipe may require added thickness and/or protective coatings. Concrete pipe may require extra thickness of concrete cover over the steel reinforcement, high density concrete, using supplementary cementitious materials, epoxy coated reinforcing steel, and/or protective coatings. Means for estimating the maintenance-free service life of pipe, and techniques for extending the useful life of pipe materials are discussed in more detail in Topic 852.

The design service life for drainage facilities for all projects should be as follows:

(1) *Culverts, Drainage Systems, and Side Drains.*

- (a) Roadbed widths greater than 28 feet - 50 years.
- (b) Greater than 10 feet of cover - 50 years.
- (c) Roadbed widths 28 feet or less and with less than 10 feet of cover - 25 years.
- (d) Installations under interim alignment - 25 years.

(2) *Overside Drains.*

- (a) Buried more than 3 feet- 50 years.
- (b) All other conditions, such as on the surface of fill slopes - 25 years.

(3) *Subsurface Drains.*

- (a) Underdrains within roadbed - 50 years.
- (b) Underdrains outside of roadbed - 25 years.
- (c) Stabilization trench drains - 50 years.

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

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

855.2 Abrasion

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

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

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

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

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

Figure 855.1
Minor Bedload Volume

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

Table 855.2D constitutes a guide for anticipated wear (in mils/year) to metal pipe by abrasive channel materials. No additional abrasion wear is anticipated for steel for the lower three abrasion levels defined in Table 855.2A, because it is assumed that there is some degree of abrasion incorporated within California Test 643 and Figure 855.3B. Figure 855.3B, "Chart for Estimating Years to Perforation of Steel Culverts," is part of a Standard California Department of Transportation Test Method derived from highway culvert investigations. This chart alone is not used for determining service life because it does not consider the effects of abrasion or overfill; it is for estimating the years to the first corrosion perforation of the wall or invert of the CSP. Additional gauge thickness or invert protection may be needed if the thickness for structural requirements (i.e., for overfill) is inadequate for abrasion potential.

Table 855.2E indicates relative abrasion resistance properties of pipe and lining materials and summarizes the findings from "Evaluations of Abrasion Resistance of Pipe and Pipe Lining Materials Final Report FHWA /CA/TL-CA01-0173 (2007)". This report may be viewed at the following web address: http://www.dot.ca.gov/new/tech/researchreports/reports/2007/evaluation_of_abrasion_resistance_final_report.pdf. See Figure 855.2.

Figure 855.2
Abrasion Test Panels

Various culvert material test panels shown in Figure 855.2 after 1 year of wear at site with moderate to severe abrasion (velocities generally exceed 13 ft/s with heavy bedload).

Table 855.2F is based on Tables 855.2D and 855.2E and constitutes a guide for selecting the minimum material thickness of abrasive resistant invert protection for various materials to achieve 50 years of maintenance-free service life.

Structural metal plate pipe and arches provide a viable option for large diameter pipes (60 inches or larger) in abrasive environments because increased thickness can be specified for the lower 90 degrees or invert plates. If the thickness for structural requirements is inadequate for abrasion potential, it is recommended to apply the increased thickness to the lower 90 degrees of the pipe only. Arches, which have a relatively larger invert area than circular pipe, generally will provide a lower abrasion potential from bedload being less concentrated.

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

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

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

Table 855.2A
Abrasion Levels and Materials

Abrasion Level	General Site Characteristics	Allowable Pipe Materials and Lining Alternatives
Level 1	• Bedloads of silts and clays or clear water with virtually no abrasive bed load. No velocity limitation.	All pipe materials listed in Table 857.2 allowable for this level. No abrasive resistant protective coatings listed in Table 855.2C needed for metal pipe.
Level 2	• Moderate bed loads of sand or gravel • Velocities ≥ 1 ft/s and ≤ 5 ft/s (See Note 1)	All allowable pipe materials listed in Table 857.2 with the following considerations: • Generally, no abrasive resistant protective coatings needed for steel pipe. • Polymeric, or bituminous coating or an additional gauge thickness of metal pipe may be specified if existing pipes in the same vicinity have demonstrated susceptibility to abrasion and thickness for structural requirements is inadequate for abrasion potential.
Level 3	• Moderate bed load volumes of sands, gravels and small cobbles. • Velocities > 5 ft/s and ≤ 8 ft/s (See Note 1)	All allowable pipe materials listed in Table 857.2 with the following considerations: • Steel pipe may need one of the abrasive resistant protective coatings listed in Table 855.2C or additional gauge thickness if existing pipes in the same vicinity have demonstrated susceptibility to abrasion and thickness for structural requirements is inadequate for abrasion potential. • Aluminum pipe may require additional gauge thickness for abrasion if thickness for structural requirements is inadequate for abrasion potential. • Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (equivalent to galv. Steel) where pH < 6.5 and resistivity < 20,000. Lining alternatives: • PVC, • Corrugated or Solid Wall HDPE, • CIPP

Note:

(1) If bed load volumes are minimal, a 50% increase in velocity is permitted.

Table 855.2A
Abrasion Levels and Materials (Con't)

Abrasion Level	General Site Characteristics	Allowable Pipe Materials and Lining Alternatives
Level 4	• Moderate bed load volumes of angular sands, gravels, and/or small cobbles/rocks. (See Note 1) • Velocities > 8 ft/s and ≤ 12 ft/s	All allowable pipe materials listed in Table 857.2 with the following considerations: • Steel pipe will typically need one of the abrasive resistant protective coatings listed in Table 855.2C or may need additional gauge thickness if thickness for structural requirements is inadequate for abrasion potential. • Aluminum pipe not recommended. • Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (wear rate equivalent to galv. steel) where pH < 6.5 and resistivity < 20,000 if thickness for structural requirements is inadequate for abrasion potential. • Increase concrete cover over reinforcing steel for RCB (invert only). RCP generally not recommended. • Corrugated HDPE (Type S) limited to ≥ 48" min. diameter. Corrugated HDPE Type C not recommended. • Corrugated PVC limited to ≥ 18" min. diameter Lining alternatives: • Closed profile or SDR 35 PVC (corrugated and ribbed PVC limited to ≥ 18" min. diameter. • SDR HDPE • CIPP (min. thickness for abrasion specified) • Concrete and authorized cementitious pipeliners and invert paving. See Table 855.2F.

Note:

(1) For minor bed load volumes, use Level 3.

Table 855.2A
Abrasion Levels and Materials (Con't)

Abrasion Level	General Site Characteristics	Allowable Pipe Materials and Lining Alternatives
Level 5	• Moderate bed load volumes of angular sands and gravel or rock (See Note 1). • Velocities > 12 ft/s and ≤ 15 ft/s	• Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (wear rate equivalent to galv. steel) where pH < 6.5 and resistivity < 20,000 if thickness for structural requirements is inadequate for abrasion potential. • For steel pipe invert lining additional gauge thickness is recommended if thickness for structural requirements is inadequate for abrasion potential. See lining alternatives below. • Increase concrete cover over reinforcing steel for RCB (invert only). RCP generally not recommended Lining alternatives: • Closed profile (≥ 42 in) or SDR 35 PVC (PVC liners not recommended when freezing conditions are often encountered and cobbles or rocks are present) • SDR HDPE • CIPP (with min. thickness for abrasion specified) • Concrete and authorized cementitious pipeliners and invert paving. See Table 855.2F.

Note:

(1) For minor bed load volumes, use Level 3.

Table 855.2A
Abrasion Levels and Materials (Con't)

Abrasion Level	General Site Characteristics	Allowable Pipe Materials and Lining Alternatives
Level 6	• Moderate bed load volumes of angular sands and gravel or rock (See Note 1). • Velocities > 15 ft/s and ≤ 20 ft/s or • Heavy bed load volumes of angular sands and gravel or rock (See Note 1). • Velocities > 12 ft/s	• Aluminized steel (type 2) not recommended without invert protection or increased gauge thickness (wear rate equivalent to galv. steel) where pH < 5.5 and resistivity < 20,000. • None of the abrasive resistant protective coatings listed in Table 855.2C are recommended for protecting steel pipe. • Invert lining and additional gauge thickness is recommended. See lining alternatives below. • Corrugated HDPE not recommended. Corrugated and closed profile PVC pipe not recommended. • RCP not recommended. Increase concrete cover over reinforcing steel recommended for RCB (invert only) for velocities up to 15 ft/s. RCB not recommended for velocities greater than 15 ft/s unless invert lining is placed (see lining alternatives below). Lining/replacement alternatives: • ≥ 27 in SDR 35 PVC (PVC liners not recommended when freezing conditions are often encountered and cobbles or rocks are present) or HDPE SDR (minimum wall thickness 2.5") • CIPP (with min. thickness for abrasion specified), • Concrete with embedded aggregate (e.g. cobbles or RSP (facing)): (for all bed load sizes a larger, harder aggregate than the bed load, decreased water cement ratio and an increased concrete compressive strength should be specified). • Alternative invert linings may include steel plate, rails or concreted RSP, and abrasion resistant concrete (Calcium Aluminate). See authorized cementitious pipeliners and invert paving in Table 855.2F. • For new/replacement construction, consider "bottomless" structures.

Note:

(1) For minor bed load volumes, use Level 3.

Table 855.2B
Bed Materials Moved by Various Flow Depths and Velocities

Bed Material	Grain Dimensions (inches)	Approximate Nonscour Velocities (feet per second) Mean Depth (feet)			
		1.3	3.3	6.6	9.8
Boulders	more than 10	15.1	16.7	19.0	20.3
Large cobbles	10 – 5	11.8	13.4	15.4	16.4
Small cobbles	5 – 2.5	7.5	8.9	10.2	11.2
Very coarse gravel	2.5 – 1.25	5.2	6.2	7.2	8.2
Coarse gravel	1.25 – 0.63	4.1	4.7	5.4	6.1
Medium gravel	0.63 – 0.31	3.3	3.7	4.1	4.6
Fine gravel	0.31 – 0.16	2.6	3.0	3.3	3.8
Very fine gravel	0.16 – 0.079	2.2	2.5	2.8	3.1
Very coarse sand	0.079 – 0.039	1.8	2.1	2.4	2.7
Coarse sand	0.039 – 0.020	1.5	1.8	2.1	2.3
Medium sand	0.020 – 0.010	1.2	1.5	1.8	2.0
Fine sand	0.010 – 0.005	0.98	1.3	1.6	1.8
Compact cohesive soils					
Heavy sandy loam		3.3	3.9	4.6	4.9
Light		3.1	3.9	4.6	4.9
Loess soils in the conditions of finished settlement		2.6	3.3	3.9	4.3

Notes:

(1) Bed materials may move if velocities are higher than the nonscour velocities.

(2) Mean depth is calculated by dividing the cross-sectional area of the waterway by the top width of the water surface. If the waterway can be subdivided into a main channel and an overbank area, the mean depths of the channel and the overbank should be calculated separately. For example, if the size of moving material in the main channel is desired, the mean depth of the main channel is calculated by dividing the cross-sectional area of the main channel by the top width of the main channel.

Table 855.2C

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

Flow Velocity (ft/s)	Channel Materials	Bituminous Coating (yrs.) (hot-dipped)	Bituminous Coating & Paved Invert (yrs.)	Polymeric Sheet Coating (yrs.)	Polyethylene (CSSRP) (yrs.)
	Non-Abrasive	8	15	*	*
≥ 1 – ≤ 8 [(1)]	Abrasive	6-0	15-2	30-5	*
> 8 – ≤ 12	Abrasive	0	2-0	5-0	70-35
> 12 – ≤ 15	Abrasive	**	**	**	35-8***
> 12 – ≤ 20	Abrasive & heavy bedloads	****	****	****	****

* Provides adequate abrasion resistance to meet or exceed a 50-year design service life.

** Abrasive resistant protective coatings not recommended, increase steel thickness to 10 gage.

*** Not recommended above 14 fps flow velocity.

**** Contact District Hydraulics Branch. See Table 855.2F.

Notes:

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

Table 855.2D

Guide for Anticipated Wear to Metal Pipe by Abrasive Channel Materials

Flow Velocity (ft/s)	Channel Materials	Anticipated Wear (mils/yr)		
		Plain Galvanized	Aluminized Steel (Type 2)	Aluminum**
	Non-Abrasive	0*	0*	0
≥ 1 – ≤ 8	Abrasive	0*	0*	0 – 1.5
> 8 – ≤ 12	Abrasive	0.5 – 1	0.5 – 1	1.5 – 3
> 12 – ≤ 15	Abrasive	1 – 3.5	1 – 3.5	3 – 10.5
> 12 – ≤ 20	Abrasive & Heavy bedloads	2.5 – 10	2.5 – 10	7.5 – 30

* Refer to California Test 643 and Figure 855.3B.

** Refer to Figure 855.3A.

Note:

1 mil = 0.001"

Table 855.2E

Relative Abrasion Resistance Properties of Pipe and Lining Materials*

Material	Relative Wear (dimensionless)
Steel	1
Aluminum	1.5 – 3
PVC	2
Polyester Resin (CIPP)	2.5 – 4
HDPE	4 – 5
Concrete (RCP 4000 – 7000 psi)	75 – 100
Calcium Aluminate (Mortar)	30-40
Calcium Aluminate (Concrete)	20 – 25
Basalt Tile	1
Polyethylene (CSSRP)	1 – 2

* Evaluation of Abrasion Resistance of Pipe and Pipe Lining Materials Final Report FHWA/CA/TL-CA01-0173 (2007).

Table 855.2F

Guide for Minimum Material Thickness of Abrasive Resistant Invert Protection to Achieve 50 Years of Maintenance-Free Service Life

Abrasion Level & Flow Velocity (ft/s)	Channel Materials	Concrete [4] (in)	Steel Pipe & Plate (in)	Aluminum Pipe & Plate (in)	PVC (in)	HDPE (in)	CIPP (in)	Calcium Aluminate Abrasion Resistant Concrete [5] (in)	Mortar [5]	
									Calcium Aluminate (in)	Geopolymer (in)
Level 4 > 8 – ≤ 12	Abrasive	2 – 4	0.052	0.075 – 0.164	0.1	0.125 – 0.25	0.1 – 0.3	(6)	1-2	2-4
Level 5 > 12 – ≤ 15	Abrasive	4 – 13	0.052 – 0.18	(2)	0.1 – 0.35	0.25 – 0.875	0.3 – 0.70	3[6]	2-5	4-13
Level 6 > 12 – ≤ 20	Abrasive & Heavy bedloads	(1)	0.109 – 0.5	(2)	0.25 – 1.0[3]	0.625 – 2.5	0.5 – 2	3 – 5	5-8	(1)

Notes:

(1) For flow velocity > 12 ft/s ≤ 14 ft/s use 9" – 15". For > 14 ft/s use CRSP or other abrasion resistant layer special design with, or in lieu of concrete or geopolymer mortar.

(2) Not recommended without invert protection.

(3) PVC liners not recommended when freezing conditions are often encountered and cobbles and rocks are present.

(4) Values shown based on RCP abrasion test results. See Table 855.2E. Results may differ from concrete specified under 15-6.04 for invert paving which must have a minimum compressive strength of 6,000 psi at 28 days and 1 ½-inch maximum grading.

(5) See Authorized Materials List for Cementitious Pipeliners and Concrete Invert Paving: http://www.dot.ca.gov/hq/esc/approved_products_list/
Standard Mortar (Section 51-1.02F of the Standard Specifications) not recommended for Abrasion Level 4 or higher.

(6) Minimum thickness recommended is 3". Not practical or economically viable for Level 4. Consider calcium aluminate mortar or standard concrete (Section 90 of the Standard Specifications) for lower range of Level 5.

culverts smaller than 30 inches or larger diameters with insignificant abrasive bedload volumes)

Abrasion resistance for any concrete lining is dependent upon the thickness, quality, strength, and hardness of the aggregate and compressive strength of the concrete as well as the velocity of the water flow coupled with abrasive sediment content and acidity. Abrasion resistant concrete or mortar made from calcium aluminate provides much improved abrasion resistance over cementitious concrete and should be considered as a viable countermeasure in extremely abrasive conditions (i.e, velocity greater than 15 feet per second with heavy bedload). See Table 855.2F.

Plastic materials typically exhibit good abrasion resistance but service life is constrained by the manufactured thickness of typical pipe profiles. Both PVC and HDPE corrugated pipe are limited for their use in moderate and heavy bedload abrasion conditions by the combined manufactured inner liner and corrugated wall thicknesses. For culvert rehabilitation, PVC and HDPE pipe slip lining products (e.g. solid wall HDPE) are viable options for applications in moderate and heavy bedload abrasion conditions (see Table 855.2A).

Table 855.2A can be used as a "preliminary estimator" of abrasion potential for material selection to achieve the required service life, however, it incorporates only three of the primary abrasion factors; bedload volume, bedload type and flow velocity and the general assumption is the materials are angular, hard and abrasive. As discussed above, the other factors that are not used in the table should also be carefully considered. For example, under similar hydraulic conditions, heavy volumes of hard, angular sand may be more abrasive than small volumes of relatively soft, large or rounded rocks. Furthermore, two sites with similar site characteristics, but different hydrologic characteristics, i.e., volume, duration and frequency of stream flow in the culvert, will likely also have different abrasion levels. Table 855.2B can be used as a guide with Table 855.2A to determine the maximum size of material that can be moved through a pipe. Field observations of channel bed material both upstream and downstream from the pipe are extremely important for estimating the size range of transportable material in the channel.

855.3 Corrosion

Corrosion is the destructive attack on a pipe by a chemical reaction with the materials surrounding the pipe. Corrosion problems can occur when metal pipes are used in locations where the surrounding materials have excess acidity or alkalinity. The relative acidity of a substance is often expressed by its pH value. The pH scale ranges from 1 to 14, with 1 representing extreme acidity, and 14 representing extreme alkalinity, and 7 representing a neutral substance. The closer the pH value is to 7, the less potential the substance has for causing corrosion.

Corrosion is an electrolytic process and requires an electrolyte (generally moisture) and oxygen to proceed. As a result, it has the greatest potential for causing damage in soils that have a relative high ability to pass electric current. The ability of a soil to convey current is expressed as its resistivity in ohm-cm, and a soil with a low resistivity has a greater ability to conduct electricity. Very dry areas (e.g., desert environments) have a limited availability of electrolyte, and totally and continuously submerged pipes have limited oxygen availability. These extreme conditions (among others) are not well represented by AltPipe, and some adjustment in the estimated service life for pipes in these conditions should be made. See Index 857.2

Corrosion can also be caused by excessive acidity in the water conveyed by the pipe. Water pH can vary considerably between watersheds and seasons.

Because failure can occur at any point along the length of the pipe (e.g. tidal zones), the designer must look at the conditions and how they may vary along the pipe length - and select for input into AltPipe those conditions that represent the most severe situation along the length.

AltPipe operates based on some fairly basic assumptions for corrosion and minimum resistivity that are part of California Test 643. Altpipe will list all viable alternatives for achieving design service life. Where enhanced soilside corrosion protection is needed, aluminum or aluminized pipe (if within acceptable pH/min. resistivity ranges), bituminous coatings or polymeric sheet coating should be considered.

Aluminum, and the aluminum coating provided by Aluminized Steel (Type 2) pipe, corrodes differently than steel and will provide adequate durability to meet the 50-year service life criterion within the acceptable pH range of 5.5-8.5 and minimum resistivity greater than 1500 ohm-cm without need for specifying a thicker gauge or additional coating, whereas under the same range galvanized steel may need a protective coating or an increase in thickness to provide a 50-year maintenance-free service life (with respect to corrosion). Figure 855.3A should be used to determine the limitations on the use of corrugated aluminum pipe for various levels of pH and minimum resistivity. The minimum thickness (0.060 inch) of aluminum pipe obtained from the chart only satisfies corrosion requirements. Overfill requirements for minimum metal thickness must also be satisfied. The metal thickness of corrugated aluminum pipe should satisfy both requirements.

Figure 855.3A should be used to determine the minimum thickness and limitation on the use of corrugated steel and spiral rib pipe for various levels of pH and minimum resistivity. For example, given a soil environment with pH and minimum resistivity levels of 6.5 and 15,000 ohm-cm, respectively, the minimum thicknesses for the various metal pipes are: 1) 0.109 inch (12 gage) galvanized steel, 2) 0.064 inch (16 gage) aluminized steel (type 2) and 3) 0.060 inch (16 gage) aluminum. The minimum thickness of metal pipe obtained from the figure only satisfies corrosion requirements. Overfill requirements for minimum metal thickness must also be satisfied. The metal thickness of corrugated pipe and steel spiral rib pipe that satisfies both requirements should be used.

Figure 855.3B, "Chart for Estimating Years to Perforation of Steel Culverts," is part of a Standard California Department of Transportation Test Method derived from highway culvert investigations. This chart alone is not used for determining service life because it does not consider the effects of abrasion or overfill; it is for estimating the years to the first corrosion perforation of the wall or invert of the CSP.

855.4 Protection of Concrete Pipe and Drainage Structures from Acids, Chlorides and Sulfates

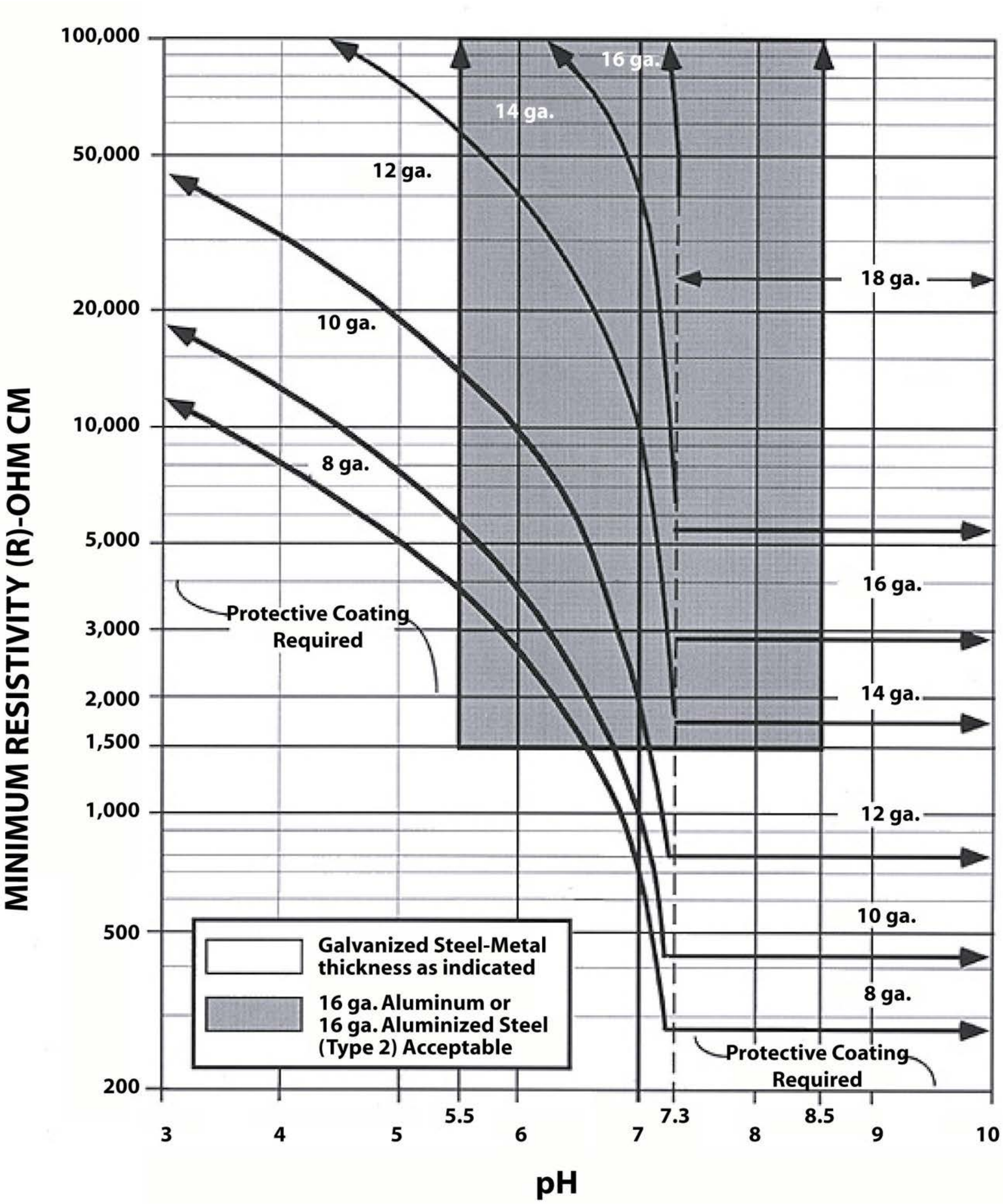
Table 855.4A indicates the limitation on the use of concrete by acidity of soil and water. Table 855.4A is also a guide for designating cementitious material restrictions and water content restrictions for various ranges of sulfate concentrations in soil and water for all cast in place and precast construction of drainage structures.

For pH ranging between 7.0 and 3.0 and for sulfate concentrations between 1500 and 15,000 ppm, concrete mix designs conforming to the recommendations given in Table 855.4A should be followed. Higher sulfate concentrations or lower pH values may preclude the use of concrete or would require the designer to develop and specify the application of a complete physical barrier. Reinforcing steel can be expected to respond to corrosive environments similarly to the steel in CSP.

Table 855.4B provides a guide for minimum concrete cover requirements for various ranges of chloride concentrations in soil and water for all precast and cast in place construction of drainage structures.

(1) RCP. In relatively severe acidic, chloride or sulfate environments (either in the soil or water) as identified in the project Materials Report, the means for offsetting the effects of the corrosive elements is to either increase the cover over the reinforcing steel, increase the cementitious material content, or reduce the water/ cementitious material ratio. The identified constituent concentration levels should be entered into AltPipe to verify what combinations of increased cover (in 1/4-inch intervals from 1 inch to a maximum of 1-1/2 inches), increased cementitious material content (in increments of 47 pounds from 470 pounds to a maximum of 564 pounds), will provide the necessary service life (typically 50 years). Per an agreement with Industry, the water to cementitious material ratio is set at 0.40. AltPipe is specifically programmed to provide RCP mix and cover designs that are compatible with industry practice, and are based on their agreements with Caltrans. For corrosive condition installations such as low pH (<4.5),

Figure 855.3A
Minimum Thickness of Metal Pipe
for 50-Year Maintenance-Free Service Life [(2)]



Notes:

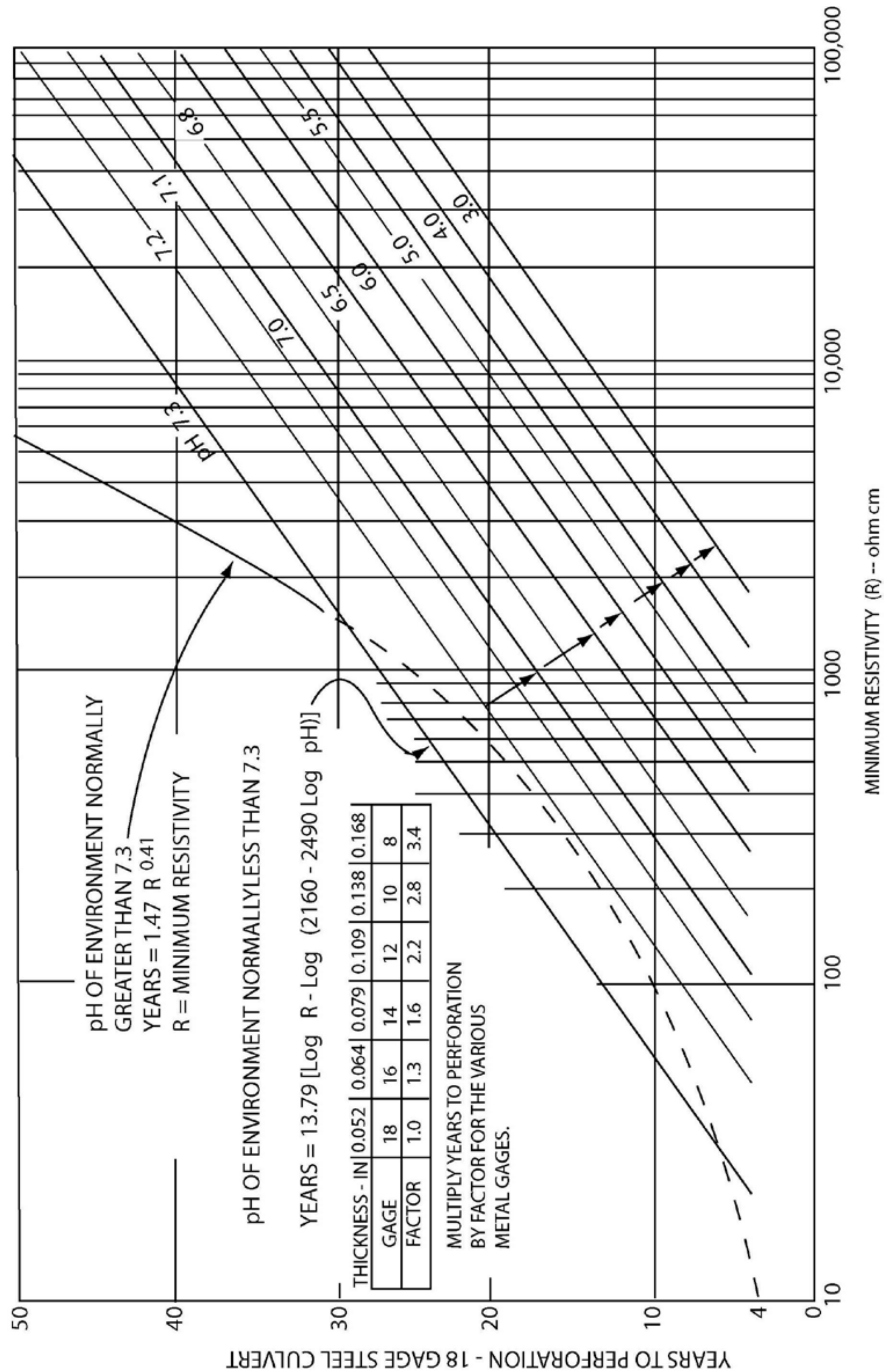
(1) For pH and aluminum resistivity levels not shown refer to Fig. 855.3B steel pipes. (California Test 643)

(2) Service life estimate are for various corrosive conditions only.

(3) Refer to Index 852.3(2) and 852.4(2) for appropriate selection of metal thickness and protection coating to achieve service life requirements.

Figure 855.3B

Chart for Estimating Years to Perforation of Steel Culverts



Chlorides (>2,000 ppm) or Sulfates (> 2,000 ppm), the following service life (SL) equation provides the basis for RCP design in AltPipe:

$$SL = 10^3 \times 1.107^{Cc} \times Cc^{0.717} \times Dc^{1.22} \times (K+1)^{-0.37} \times W^{-0.631} - 4.22 \times 10^{10} \times pH^{-14.1} - 2.94 \times 10^{-3} \times S + 4.41$$

Where: S = Environmental sulfate content in ppm.

Cc = Sacks of cement (94 lbs each) per cubic yard of concrete.

Dc = Concrete cover in inches.

K = Environmental chloride concentration in ppm.

W = Water by volume as percentage of total mix.

pH = The measure of relative acidity or alkalinity of the soil or water. See Index 855.3.

Where the measured concentration of chlorides exceeds 2000 ppm for RCP that is placed in brackish or marine environments and where the high tide line is below the crown of the invert, the AltPipe input for chloride concentration will default to 25,000 ppm.

Contact the District Materials unit or the Corrosion Technology Branch in DES for design recommendations when in extremely corrosive conditions. Non-Reinforced concrete pipe is not affected by chlorides or stray currents and may be used in lieu of RCP with additional concrete cover and/or protective coatings for sizes 36" in diameter and smaller. See Index 852.1(4) and Table 855.4A. Where conditions occur that RCP designs as produced by AltPipe will not work, the Office of State Highway Drainage Design within the Division of Design should be contacted.

855.5 Material Susceptibility to Fire

Fire can occur almost anywhere on the highway system. Common causes include forest, brush or grass fires that either enter the right-of-way or begin within it. Less common causes include spills of flammable liquids that ignite or vandalism. Storm drains, which are completely buried would typically be impacted by spills or vandalism. Because these are such low probability events, prohibitions on material placement for storm drains are not typically warranted.

Cross culverts and exposed overside drains are the placement types most subject to burning or melting and designers should consider either limiting the alternative pipe listing to non-flammable pipe materials or providing a non-flammable end treatment to provide some level of protection.

. Plastic pipe and pipes with coatings (typically of bituminous or plastic materials) are the most susceptible to damage from fire. Of the plastic pipe types which are allowed, PVC will self extinguish if the source of the fire is eliminated (i.e., if the grass or brush is consumed or removed) while HDPE can continue to burn as long as an adequate oxygen supply is present. Based on testing performed by Florida DOT, this rate of burning is fairly slow, and often self extinguished if the airflow was inhibited (i.e., pipe not aligned with prevailing wind or ends sheltered from air flow).

Due to the potential for fire damage, plastic pipe is not recommended for overside drain locations where there is high fire potential (large amounts of brush or grass or areas with a history of fire) and where the overside drain is placed or anchored on top of the slope.

Where similar high fire potential conditions exist for cross culverts, the designer may consider limiting the allowable pipe materials indicated on the alternative pipe listing to non-flammable material types, use concrete endwalls that eliminate exposure of the pipe ends, or require that the end of flammable pipe types be replaced with a length of non-flammable pipe material.

Topic 856 - Height of Fill

An essential aspect of pipe selection is the height of fill/cover over the pipe. This cover dissipates live loads from traffic, both during construction and after the facility is open to the public.

856.1 Construction Loads

See Standard Plan D88 for table of minimum cover for construction loads.

Table 855.4A

Guide for the Protection of Cast-In-Place and Precast Reinforced and Unreinforced Concrete Structures[5] Against Acid and Sulfate Exposure Conditions[1],[2]

Soil or Water pH	Sulfate Concentration of Soil or Water (ppm)	Cementitious Material Requirements [3]	Water Content Restrictions
7.1 to 14	0 to 1,499	Standard Specifications Section 90	No Restrictions
5.6 to 7.0	1,500 to1,999	Standard Specifications Section 90	Maximum water-to-cementitious material ratio of 0.45
3 to 5.5[4]	2,000 to 15,000[4]	675 lb/cy minimum: Type II or Type V portland cement and required supplementary cementitious materials per Standard Specification 90-1.02H	Maximum water-to-cementitious material ratio of 0.40

Notes:

(1) Recommendations shown in the table for the cementitious material requirements and water content restrictions should be used if the pH and/or the sulfate conditions in Column 1 and/or Column 2 exists. Sulfate testing is not required if the minimum resistivity is greater than 1,000 ohm-cm.

(2) The table lists soil/water pH and sulfate concentration in increasing level of severity starting from the top of the table. If the soil/water pH and the sulfate concentration are at different levels of severity, the recommendation for the more severe level will apply. For example, a soil with a pH of 4.0, but with a sulfate concentration of only 1,600 ppm would require a minimum of 675 lb/cy of cementitious material. The maximum water-to-cementitious material ratio would be 0.40.

(3) Cementitious material shall conform to the provisions in Section 90 of the Standard Specifications.

(4) Additional mitigation measures will be needed for conditions where the pH is less than 3 and/or the sulfate concentration exceeds 15,000 ppm. Mitigation measures may include additional concrete cover and/or protective coatings. For additional assistance, contact the Corrosion Technology Branch of Materials Engineering and Testing Services (METS) at 5900 Folsom Boulevard Sacramento, CA. 95819.

(5) Does not include RCP.

Table 855.4B

Guide for Minimum Cover Requirements for Cast-In-Place and Precast Reinforced Concrete Structures[3] for 50-Year Design Life in Chloride Environments

Chloride Concentration (ppm)			
500 to 2000	2001 to 5000	5001 to 10000	10000 +
1.5 in.[1]	2.5 in.[1]	3 in.[1]	4 in.[1]
1.5 in.[2]	1.5 in.[2]	2 in.[2]	3 in.[2]

Notes:

(1) Supplementary cementitious materials are required. Typical minimum requirement consists of 675#/cy minimum cementitious material with 75% by weight of Type II or Type V portland cement and 25% by weight of either fly ash or natural pozzolan. A maximum w/cm ratio of 0.40 is specified. Fly ash or natural pozzolan may have a CaO content of up to 10%. Section 90-1.02B(3) of the Standard Specifications provides requirements.

(2) Additional supplementary cementitious materials per the requirements of Section 90-1.02B(3) of the Standard Specifications are required in order to achieve the listed reduction in concrete cover.

(3) Does not include RCP.

856.2 Concrete Pipe, Box and Arch Culverts

(1) *Reinforced Concrete Pipe.* See Standard Plan A62D and A62DA for the maximum height of overfill for reinforced concrete pipe, up to and including 120-inch diameter (or reinforced oval pipe and reinforced concrete pipe arch with equivalent cross-sectional area), using the backfill method or type shown. For oval shaped reinforced concrete pipe fill heights, see Standard Plan A62D and Indirect Design D-Load (Marsten/Spangler Method). Allowable cover for oval shaped reinforced concrete pipe is determined by using Method 2 (Note 8). See Standard Plan D79 and D79A for pre-cast reinforced concrete pipe Direct Design Method (pertains to circular pipe only).

The designer should be aware of the premises on which the tables on Standard Plan A62D, A62DA, D79 and D79A are computed as well as their limitations. The cover presupposes:

- That the bedding and backfill satisfy the terms of the Standard Specifications, the conditions of cover and pipe size required by the plans, and take into account the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert equal in magnitude to that of the adjoining material outside the trench.
- Subexcavation and backfill as required by the Standard Specifications where unyielding foundation material is encountered.

If the height of overfill exceeds the tabular values on Standard Plan A62D and A62DA a special design is required; see Index 829.2.

(1) *Concrete Box and Arch Culverts.* Single and multiple span reinforced concrete box culverts are completely detailed in the Standard Plans. For cast-in-place construction, strength classifications are shown for 10 feet and 20 feet overfills. See Standard Plan numbers D80, D81 and D82. Pre-cast reinforced concrete box culverts require a minimum of 1 foot overfill and limit fill height to 12 feet maximum. See Standard Plans D83A, D83B and A62G. For fill height design criteria for CIP Bottomless 3-sided rigid frame culverts see XS-Sheets 17-050-1, 2, 3, 4 and 5. Cast-in-place reinforced concrete arch culverts are no longer economically feasible structures and last appeared in the 1997 Standard Plans. Questions regarding fill height for concrete arch culverts or extensions should be directed to the Underground Structures Branch of DES - Structures Design.

856.3 Metal Pipe and Structural Plate Pipe

Basic Premise - To properly use the fill height design tables, the designer should be aware of the premises on which the tables are based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications and Standard Plan A62F, the conditions of cover, and pipe size required by the plans and the essentials of Index 829.2.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.

Limitations - In using the tables, the following restrictions must be kept in mind:

- The values given for each size of pipe constitute the maximum height of overfill or cover over the pipe for the thickness of metal and kind of corrugation.
- The thickness shown is the structural minimum. Where abrasive conditions are anticipated, additional metal thickness or invert treatments as stated under Index 852.4(5) and Index 852.6(2)(c) should be provided when required to fulfill the design service life requirements of Topic 855.
- Where needed, adequate provisions for corrosion resistance must be made to achieve the required design service life called for in the references mentioned herein.
- Table 856.3D shows the limit of heights of cover for corrugated steel pipe arches based on the supporting soil sustaining a factored bearing pressure varying between 3.38 tons per square feet to 3.55 tons per square feet. Table 856.3J shows similar values for corrugated aluminum pipe arches.

- The values given for each size of structural plate pipe or arch constitute the maximum height of overfill or cover over the pipe or arch for the thickness of metal and kind of corrugation.
- Tables 856.3N & P show the limit of heights of cover for structural plate arches based on the supporting soil sustaining a factored bearing pressure of 6 tons per square foot at the corners.

Special Designs.

- If the height of overfill exceeds the tabular values, or if the foundation investigation reveals that the supporting soil will not develop the bearing pressure on which the overfill heights for pipe arches are based, a special design prepared by DES - Structures Design is required. See index 829.2.
- Non-standard pipe diameters and arch sizes are available. Loading capacity of special designs needs to be verified with the Underground Structures Branch of DES - Structures Design.
- Aluminum pipe fill height tables are based on use of H-32 temper aluminum. If use of aluminum is necessary and greater structural capacity is required, H-34 temper can be specified. Contact Underground Structures branch of DES-Structures Design for calculation of allowable fill height.

(1) *Corrugated Steel Pipe and Pipe Arches, Steel Spiral Rib Pipe, Structural Steel Plate Pipe and Structural Steel Plate Pipe Arches.* The allowable overfill heights for corrugated steel pipe and pipe arches for the various diameters or arch sizes and metal thickness are shown on Tables 856.3A, B, C & D. For steel spiral rib pipe, overfill heights are shown on Tables 856.3E, F, G & H. Table 856.3G gives the allowable overfill height for composite steel spiral rib pipe.

For structural steel plate pipe and structural steel plate pipe arches, overfill heights are shown on Tables 856.3M & N. For maximum height of fill over structural steel plate vehicular undercrossings, see Standard Plan B14-1.

(2) *Corrugated Aluminum Pipe and Pipe Arches, Aluminum Spiral Rib Pipe and Structural Aluminum Plate Pipe and Structural Aluminum Plate Pipe Arches.* The allowable overfill heights for corrugated aluminum pipe and pipe arches for various diameters and metal thickness are shown on Tables 856.3H, I & J. For aluminum spiral rib pipe, overfill heights are shown on Tables 856.3K & L.

For structural aluminum plate pipe and structural aluminum plate pipe arches, overfill heights are shown on Tables 856.3O, & P.

856.4 Plastic Pipe

The allowable overfill heights for plastic pipe for various diameters are shown in Tables 856.4 and 856.5. To properly use the plastic pipe height of fill table, the designer should be aware of the basic premises on which the table is based as well as their limitations. The design tables presuppose:

- That bedding and backfill satisfy the terms of the Standard Specifications and Standard Plan A62F, the conditions of cover, and pipe size required by the plans and the essentials of Index 829.2.
- That corrugated high density polyethylene (HDPE) pipe that is greater than 48" in size shall be backfilled with cementitious (slurry cement, CLSM or concrete) backfill.
- That where cementitious or flowable backfill is used for structural backfill, the backfill shall be placed to a level not less than 12 inches above the crown of the pipe.
- That a small amount of settlement will occur under the culvert, equal in magnitude to that of the adjoining material outside the trench.
- That the average water table elevation is at or below the pipe springline.
- Corrugated HDPE pipe, Type C is recommended for placement only outside the roadbed where vehicular loading is unlikely (e.g., overside drains, medians) unless cementitious backfill is specified.

856.5 Minimum Height of Cover

Table 856.5 gives the minimum thickness of cover required for design purposes over pipes and pipe arches. For construction purposes, a minimum cover of 6 inches greater than the roadway structural section is desirable for all types of pipe.

Table 856.3A

Corrugated Steel Pipe
Helical Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)					
	Metal Thickness (in)					
	0.052 (18 ga.)	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)	0.168 (8 ga.)
	2⅔" x ½" Corrugations					
12-15	118	148	177	--	--	--
18	99	124	148	207	--	--
21	85	106	132	177	--	--
24	74	93	116	155	200	245
30	59	74	93	130	160	195
36	49	62	77	108	139	163
42	42	53	66	93	119	139
48	--	46	58	81	104	128
54	--	--	51	72	93	113
60	--	--	--	65	83	102
66	--	--	--	--	76	93
72	--	--	--	--	70	85
78	--	--	--	--	--	75
84	--	--	--	--	--	65
	3" x 1" Corrugations					
48	--	53	67	93	120	147
54	--	47	59	83	107	131
60	--	42	53	75	96	118
66	--	39	48	68	87	107
72	--	35	44	62	80	98
78	--	33	41	57	74	91
84	--	30	38	53	69	84
90	--	28	35	50	64	78
96	--	--	33	47	60	74
102	--	--	31	44	56	69
108	--	--	--	41	53	65
114	--	--	--	39	50	62
120	--	--	--	37	48	59

Table 856.3B

Corrugated Steel Pipe
Helical Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)			
	Metal Thickness (in)			
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)
	5" x 1" Corrugations			
48	47	59	83	--
54	42	53	74	95
60	38	47	66	86
66	34	43	60	78
72	31	39	55	71
78	29	36	51	66
84	27	34	47	61
90	25	31	44	57
96	--	29	41	53
102	--	28	39	50
108	--	--	37	47
114	--	--	35	45
120	--	--	33	43

Table 856.3C

Corrugated Steel Pipe
2⅔" x ½" Annular Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)				
	Metal Thickness (in)				
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)	0.168 (8 ga.)
18	54	--	--	--	--
21	46	--	--	--	--
24	40	44	--	--	--
30	32	35	--	--	--
36	27	29	38	--	--
42	30	41	65	68	--
48	26	36	57	59	62
54	--	32	50	53	55
60	--	--	45	47	50
66	--	--	--	43	45
72	--	--	--	39	41
78	--	--	--	--	38
84	--	--	--	--	35

Table 856.3D

Corrugated Steel Pipe Arches
2⅔" x ½" Helical or Annular Corrugations

			MAXIMUM HEIGHT OF COVER (ft)			
			Metal Thickness (in)			
Span-Rise (in)	Factored Bearing Demand (tons/ft^2)	Minimum Corner Radius (in)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)	0.168 (8 ga.)
21 x 15	3.50	4 1/8	10	--	--	--
24 x 18	3.38	4 7/8	10	--	--	--
28 x 20	3.49	5 1/2	10	--	--	--
35 x 24	3.49	6 7/8	10	--	--	--
42 x 29	3.49	8 1/4	10	--	--	--
49 x 33	3.49	9 5/8	10	--	--	--
57 x 38	3.55	11	--	10	--	--
64 x 43	3.54	12 3/8	--	10	--	--
71 x 47	3.54	13 3/4	--	--	10	--
77 x 52	3.49	15 1/8	--	--	--	10
83 x 57	3.45	16 1/2	--	--	--	10

Note:
(1) Cover limited by corner soil bearing pressure as shown.

Table 856.3E

Steel Spiral Rib Pipe
¾" x 1" Ribs at 11½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)		
	Metal Thickness (in)		
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)
24	44	62	105
30	36	50	84
36	30	42	70
42	25	36	60
48	22	31	53
54	20	28	47
60	--	25	42
66	--	22	38
72	--	21	35
78	--	--	32
84	--	--	30
90	--	--	28
96	--	--	--

Table 856.3F

Steel Spiral Rib Pipe
¾" x 1" Ribs at 8½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)		
	Metal Thickness (in)		
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)
24	59	83	137
30	48	66	110
36	40	55	92
42	34	47	78
48	30	41	69
54	26	37	61
60	24	33	55
66	21	30	50
72	20	27	46
78	--	25	42
84	--	23	39
90	--	--	36
96	--	--	34
102	--	--	32
108	--	--	30
114	--	--	--

Table 856.3G

Steel Spiral Rib Pipe
¾” x ¾" Ribs at 7½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)			
	Metal Thickness (in)			
	0.064 (16 ga.)	0.079 (14 ga.)	0.109 (12 ga.)	0.138 (10 ga.)
24	61	85	141	205
30	49	68	113	164
36	40	57	94	137
42	35	48	81	117
48	30	42	71	103
54	27	38	63	91
60	--	34	57	82
66	--	31	51	75
72	--	--	47	68
78	--	--	43	63
84	--	--	40	59
90	--	--	--	55

Table 856.3H
Corrugated Aluminum Pipe
Annular Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft) Metal Thickness (in)				
	0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)	0.135 (10 ga.)	0.164 (8 ga.)
	2⅔" x ½" Corrugations				
12	43	43	--	--	--
15	35	34	60	--	--
18	29	29	50	--	--
21	25	25	43	--	--
24	21	21	37	39	--
30	--	17	30	31	--
36	--	14	25	26	--
42	--	--	43	45	--
48	--	--	38	40	41
54	--	--	34	35	36
60	--	--	--	32	33
66	--	--	--	--	30
72	--	--	--	--	27
	3" x 1" Corrugations				
30	32	40	54	81	--
36	26	33	45	68	88
42	23	28	39	58	75
48	20	25	34	51	66
54	17	22	30	45	59
60	16	20	27	41	53
66	14	18	24	37	48
72	13	16	22	34	44
78	--	15	21	31	40
84	--	--	19	29	38
90	--	--	18	27	35
96	--	--	17	25	33
102	--	--	--	24	31
108	--	--	--	22	29
114	--	--	--	--	28
120	--	--	--	--	26

Table 856.3I
Corrugated Aluminum Pipe
Helical Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)				
	Metal Thickness (in)				
	0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)	0.135 (10 ga.)	0.164 (8 ga.)
	2⅔" x ½" Corrugations				
12	112	140	--	--	--
15	90	112	156	--	--
18	75	93	130	--	--
21	64	80	112	--	--
24	56	70	98	126	--
30	--	56	78	101	--
36	--	47	65	84	--
42	--	--	56	72	--
48	--	--	49	63	77
54	--	--	43	56	68
60	--	--	--	46	58
66	--	--	--	--	47
72	--	--	--	--	37
	3" x 1" Corrugations				
30	51	65	90	121	--
36	43	54	75	101	118
42	37	46	64	86	102
48	32	40	56	76	89
54	28	36	50	67	79
60	26	32	45	60	71
66	23	29	41	55	65
72	21	27	37	50	59
78	--	25	35	46	55
84	--	--	32	43	51
90	--	--	30	40	47
96	--	--	28	38	44
102	--	--	--	35	42
108	--	--	--	33	39
114	--	--	--	--	36
120	--	--	--	--	32

Table 856.3J

Corrugated Aluminum Pipe Arches
2⅔" x ½" Helical or Annular Corrugations

Span-Rise (in)	Factored Bearing Demand (tons/ft^2)	Minimum Corner Radius (in)	MAXIMUM HEIGHT OF COVER (ft) Metal Thickness (in) 0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)	0.135 (10 ga.)	0.164 (8 ga.)
17 x 13	3.34	3 1/2	10	--	--	--	--
21 x 15	3.49	4 1/8	10	--	--	--	--
24 x 18	3.38	4 7/8	10	--	--	--	--
28 x 20	3.49	5 1/2	--	10	--	--	--
35 x 24	3.49	6 7/8	--	10	--	--	--
42 x 29	3.49	8 1/4	--	--	10	--	--
49 x 33	3.49	9 5/8	--	--	10	--	--
57 x 38	3.55	11	--	--	--	10	--
64 x 43	3.54	12 3/8	--	--	--	10	--
71 x 47	3.54	13 3/4	--	--	--	--	10

Note:
(1) Cover is limited by corner soil bearing pressure as shown.

Table 856.3K

Aluminum Spiral Rib Pipe
¾" x 1” Ribs at 11½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)		
	Metal Thickness (in)		
	0.060 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)
24	22	31	50
30	18	24	40
36	15	20	33
42	--	17	29
48	--	--	25
54	--	--	22
60	--	--	20
66	--	--	--
72	--	--	--

Table 856.3L

Aluminum Spiral Rib Pipe
¾" x ¾" Ribs at 7½" Pitch

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)		
	Metal Thickness (in)		
	0.60 (16 ga.)	0.075 (14 ga.)	0.105 (12 ga.)
24	30	41	66
30	24	33	53
36	20	27	44
42	--	23	38
48	--	--	33
54	--	--	29
60	--	--	26
66	--	--	--
72	--	--	--

Table 856.3M
Structural Steel Plate Pipe
6" x 2" Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)							
	Metal Thickness (in)							
	0.110 (12 ga.)	0.140 (10 ga.)	0.170 (8 ga.)	0.218 (5 ga.)	0.249 (3 ga.)	0.280 (1 ga.)	0.318 (0 ga.)	0.380 (000 ga.)
60	42	60	79	105	128	140	223	268
66	38	55	71	99	116	127	203	243
72	35	50	65	91	107	116	186	223
77	32	47	61	85	100	109	174	209
84	30	43	56	78	92	100	160	192
90	28	40	52	72	85	93	149	179
96	26	37	49	68	80	87	140	168
102	24	35	46	64	75	82	132	158
108	23	33	44	60	71	78	124	149
114	22	31	41	57	67	74	118	141
120	21	30	39	54	64	70	112	134
126	20	28	37	52	61	67	107	128
132	19	27	36	49	58	63	102	122
138	18	26	34	47	56	61	91	117
144	17	25	33	45	53	58	93	112
150	16	24	31	43	51	56	89	108
156	16	23	30	42	49	54	86	103
162	15	22	29	40	47	52	83	100
168	15	21	28	39	46	50	80	96
174	14	20	27	37	44	48	77	93
180	14	20	26	36	43	46	75	90
186	13	19	25	35	41	45	72	87
192	--	18	24	34	40	44	70	84
198	--	18	24	33	39	42	68	81
204	--	17	23	32	38	41	66	79
210	--	17	22	31	36	40	64	77
216	--	--	22	30	35	39	62	75
222	--	--	21	29	34	38	60	73
228	--	--	20	28	34	37	59	71
234	--	--	20	28	33	36	57	69
240	--	--	--	27	32	35	56	67
246	--	--	--	26	31	34	54	65
252	--	--	--	26	30	33	53	64

Table 856.3N

Structural Steel Plate Pipe Arches
6" x 2" Corrugations

MAXIMUM HEIGHT OF COVER (ft)			
Span	Rise	Factored Corner Soil Bearing – 6 tons/ft^2	
		Metal Thickness (in)	
		0.110 (12 ga.)	0.140 (10 ga.)
		18" Corner Radius	
6'-1"	4'-7"	21	--
7'-0"	5'-1"	18	--
7'-11"	5'-7"	16	--
8'-10"	6'-1"	14	--
9'-9"	6'-7"	13	--
10'-11"	7'-1"	12	--
		31" Corner Radius	
13'-3"	9'-4"	17	--
14'-2"	9'-10"	16	--
15'-4"	10'-4"	13	--
16'-3"	10'-10"	12	--
17'-2"	11'-4"	12	--
18'-1"	11'-10"	11	--
19'-3"	12'-4"	--	10
19'-11"	12'-10"	--	10
20'-7"	13'-2"	--	10

NOTES:
(1) For intermediate sizes, the depth of cover may be interpolated.
(2) The 31-inch corner radius arch should be specified when conditions will permit it use.

Table 856.3O

Structural Aluminum Plate Pipe
9" x 2½" Corrugations

Diameter (in)	MAXIMUM HEIGHT OF COVER (ft)						
	Metal Thickness (in)						
	0.100	0.125	0.150	0.175	0.200	0.225	0.250
60	27	40	52	62	71	81	90
66	24	36	48	56	65	73	82
72	22	33	44	51	59	67	75
77	21	31	41	48	55	63	70
84	19	28	37	44	51	58	64
90	18	26	35	41	47	54	60
96	17	25	33	38	44	50	56
102	16	23	31	36	42	47	53
108	15	22	29	34	39	45	50
114	14	21	27	32	37	42	47
120	13	20	26	31	35	40	45
126	13	19	25	29	34	38	43
132	12	18	24	28	32	36	41
138	11	17	23	27	31	35	39
144	--	16	22	25	29	33	37
150	--	16	21	24	28	32	36
156	--	15	20	23	27	31	35
162	--	--	19	23	26	30	33
168	--	--	18	22	25	29	32
174	--	--	18	21	24	28	31
180	--	--	--	20	23	27	30
186	--	--	--	20	23	26	29
192	--	--	--	--	22	25	28
198	--	--	--	--	21	24	27
204	--	--	--	--	--	23	26
210	--	--	--	--	--	23	26
216	--	--	--	--	--	22	25
222	--	--	--	--	--	--	24
228	--	--	--	--	--	--	23

Table 856.3P

Structural Aluminum Plate Pipe Arches
9" x 2½" Corrugations

		MAXIMUM HEIGHT OF COVER (ft)					
		Factored Corner Soil Bearing – 6 tons/ft^2					
		Metal Thickness (in)					
Span	Rise	0.100	0.125	0.150	0.175	0.200	0.225
6'-7"	5'-8"	20	--	--	--	--	--
7'-9"	6'-0"	17	--	--	--	--	--
8'-10"	6'-4"	15	--	--	--	--	--
9'-11"	6'-8"	13	--	--	--	--	--
10'-3"	6'-9"	13	19	--	--	--	--
11'-1"	7'-0"	12	18	20	--	--	--
12'-3"	7'-3"	11	16	18	--	--	--
12'-11"	7'-6"	10	15	17	--	--	--
13'-1"	8'-2"	10	15	17	--	--	--
13'-11"	8'-5"	9	14	16	--	--	--
14'-0"	8'-7"	9	14	16	--	--	--
14'-8"	9'-8"	--	13	15	--	--	--
15'-7"	10'-2"	--	12	13	--	--	--
16'-1"	10'-4"	--	12	13	--	--	--
16'-9"	10'-8"	--	--	12	--	--	--
17'-9"	11'-2"	--	--	--	11	--	--
18'-8"	11'-8"	--	--	--	11	--	--
19'-10"	12'-1"	--	--	--	--	10	--
20'-10"	12'-7"	--	--	--	--	--	9
21'-6"	12'-11"	--	--	--	--	--	9

Note:
(1) 31 inch Corner Radius

Where cover heights above culverts are less than the values shown in Table 856.5, stress reducing slab details available from the Headquarters Design drainage detail library using the following web address may be used: http://onramp.dot.ca.gov/hq/design/drainage/library.php

Where cover heights are less than the values shown in the stress reducing slab details, contact Office of State Highway Drainage Design or the Underground Structures Branch of DES - Structures Design.

Topic 857 - Alternate Materials

857.1 Basic Policy

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

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

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

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

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

Table 856.4

Thermoplastic Pipe Fill Height Tables

High Density Polyethylene (HDPE) Corrugated Pipe - Type S

Size (in)	Maximum Height of Cover (ft)
12	15
15	15
18	15
24	15
30	15
36	15
42	15
48	15
54	15
60	15

High Density Polyethylene (HDPE) Corrugated Pipe - Type C

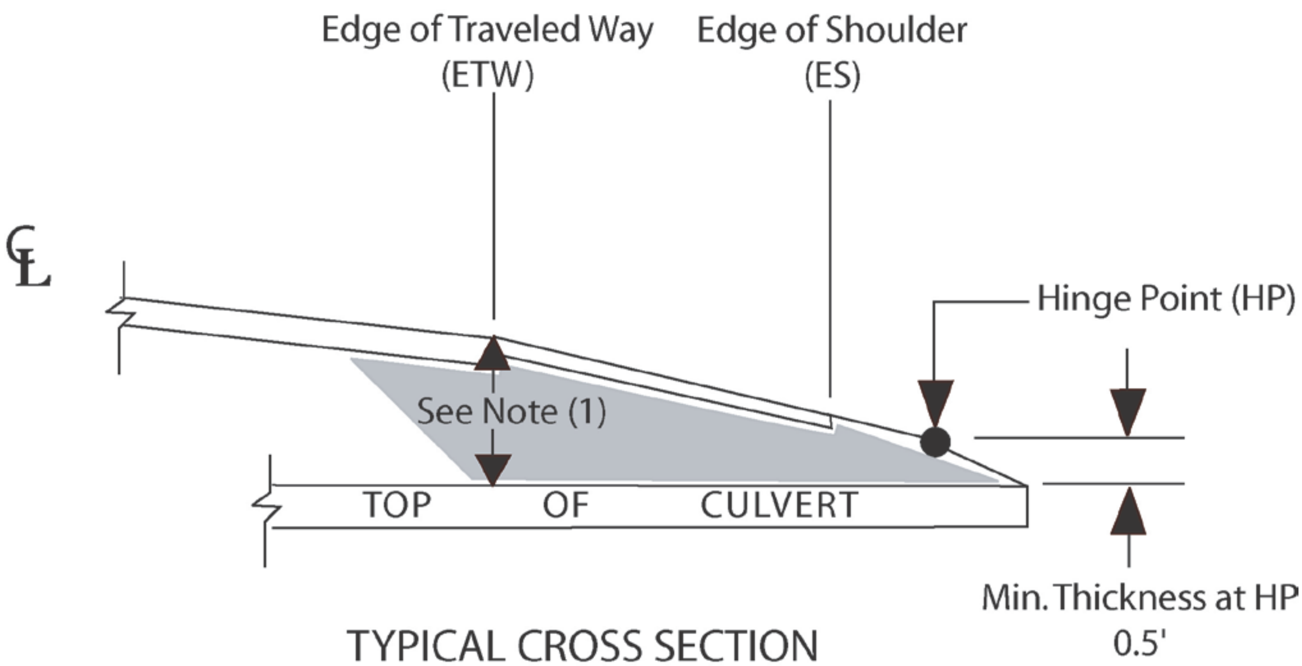
Size (in)	Maximum Height of Cover (ft)
12	5
15	5
18	5
24	5

Polyvinyl Chloride (PVC) Corrugated Pipe with Smooth Interior

Size (in)	Maximum Height of Cover (ft)
12	35
15	35
18	35
21	35
24	35
30	35
36	35

Table 856.5

Minimum Thickness of Cover for Culverts



MINIMUM THICKNESS OF COVER AT ETW							
Corrugated Metal Pipes and Pipe Arches	Steel Spiral Rib Pipe	Aluminum Spiral Rib Pipe, S ≤ 48"	Aluminum Spiral Rib Pipe, S > 48"	Structural Plate Pipe	Reinforced Concrete Pipe (RCP) Under Rigid Pavement	RCP Under Flexible Pavement or Unpaved	Plastic Pipes
S/8 or 24" Min.	S/4 or 24" Min.	S/2 or 24" Min.	S/2.75 or 24" Min.	S/8 or 24" Min.	12" Min.	(Max Outside Dimension)/8 or 24" Min.	S/2 or 24" Min.

Notes:
(1) Minimum thickness of cover is measured at ultimate or failure edge of traveled way.
(2) Table is for HL-93 live load conditions only.
(3) ”S” in the table is the maximum inside diameter or span of a section.

conditions, or critical structural and construction requirements.

- For individual drainage systems such as roadway drainage systems or culverts which operate under hydrostatic pressure or culverts governed by hydraulic considerations and which would require separate design for each culvert type.
- When alterations or extensions of existing systems are required, the culvert type may be selected to match the type used in the existing system.

857.2 Alternative Pipe Culvert Selection Procedure Using AltPipe

These instructions are general guidelines for alternative pipe culvert selection using the AltPipe computer program that is located on the Headquarters Division of Design alternative pipe culvert selection website at the following web address: http://www.dot.ca.gov/hq/oppd/altpipe.htm.

AltPipe is a web-based tool that may be used to assist materials engineers and designers in the appropriate selection of pipe materials for culvert and storm drain applications. The computations performed by AltPipe are based on the procedures and California Test Methods described in this

Chapter. AltPipe is not a substitute for the appropriate use of engineering judgment as conditions and experience would warrant. AltPipe establishes uniform procedures to assist the designer in carrying out the majority of the alternative pipe culvert selection functions of the Department, and is neither intended as, nor does it establish, a legal standard for these functions. Implementation of the results and output of this program is solely at the discretion of the user. The user is encouraged to first read the two informational links on the website titled 'Get More Information' and 'How to use Altpipe' prior to using the program.

Each alternative material selected for a drainage facility must provide the required design service life based on physical and structural factors, be of adequate size to satisfy the hydraulic design, and require the minimum of maintenance and construction cost for each site condition.

Step 1. Obtain the results of soil and water pH, resistivity, sulfate and chloride tests, proposed design life of culverts and make determination if any of the outfalls are in salty or brackish water. The Materials Report should include proposed design life and recommendations for pipe material alternatives. See Indexes 114.2 (3) and 114.3 (2).

Step 2. Obtain hydraulic studies and location data for pipe minimum sizes, and expected Q2-5 flow velocities. For pipes operating under outlet control, a critical element of pipe selection is the Manning's internal roughness value used in the hydraulic design. It is important to independently verify the roughness used in the design is applicable for the selected alternate materials from AltPipe. Rougher pipes may require larger sizes to provide adequate hydraulic capacities and need steeper slopes to produce desired cleaning velocities, usually however, pipe slope is maintained, and the only variable provided on the plans is pipe size.

Step 3. Determine the abrasion level from Table 852.2A from the maximum size of material that can be moved through a pipe, the expected Q2-5 flow velocities, and Table 855.2B. Field observations of channel bed material both upstream and downstream are recommended.

Step 4. Determine the maximum fill height.

Step 5. Using the AltPipe computer program that is located on the Headquarters Division of Design alternative pipe culvert selection website enter:

- Pipe diameter
- Maximum fill height
- Design service life
- pH
- Minimum resistivity
- Sulfate concentration
- Chloride concentration (for values greater than 2000, check boxes if end of culvert is exposed to brackish conditions and high tide line is below the crown of the culvert)
- Abrasion level
- 2-5-year Storm Flow Velocity (ft/sec)

Repeat step 5 as necessary and save each pipe in worksheet as needed and go to the final summary upon completion.

Table 857.2
Allowable Alternative Materials

Type of Installation	Service Life (yrs)[1]	Allowable Alternatives	Joint Type: Standard	Joint Type: Positive	Joint Type: Downdrain
Culverts & Drainage Systems	50	ASSRP, ASRP, CAP, CASP, CSSRP, CIPCP, CSP, NRCP, SAPP, SSPP, SSRP, RCP, RCB, PPC	X	X	--
Overside Drains	50	CAP, CASP, CSP, PPC	--	--	X
Underdrains	50	PAP, PSP, PPET, PPVCP	X	--	--
Arches (Culverts & Drainage Systems)	50	ACSPA, CAPA, CSPA, RCA, SAPPA, SSPPA, SSPA	X	X	--

<u>LEGEND</u>

ACSPA - Aluminized Corrugated Steel Pipe Arch
ASSRP - Aluminized Steel Spiral Rib Pipe
ASRP - Aluminum Spiral Rib Pipe
CAP - Corrugated Aluminum Pipe
CAPA - Corrugated Aluminum Pipe Arch
CSSRP - Composite Steel Spiral Rib Pipe
CASP - Corrugated Aluminized Steel Pipe, Type 2
CIPCP - Cast-in-Place Concrete Pipe
CSP - Corrugated Steel Pipe
CSPA - Corrugated Steel Pipe Arch
NRCP - Non-Reinforced Concrete Pipe
PAP - Perforated Aluminum Pipe
PPC - Plastic Pipe Culvert
PPET - Perforated Polyethylene Tubing
PPVCP - Perforated Polyvinyl Chloride Pipe
PSP - Perforated Steel Pipe
RCA - Reinforced Concrete Arch
RCB - Reinforced Concrete Box
RCP - Reinforced Concrete Pipe
SAPP - Structural Aluminum Plate Pipe
SAPPA - Structural Aluminum Plate Pipe Arch
SSPA - Structural Steel Plate Arch
SSPP - Structural Steel Plate Pipe
SSPPA - Structural Steel Plate Pipe Arch
SSRP - Steel Spiral Rib Pipe
X - Permissible Joint Type for the Type of installation Indicated

NOTE:
1. The design service life indicated for the various types of installations listed in the table may be reduced to 25 years in certain situations. Refer to Index 855.1 for a discussion of service life requirements.

Step 6. The following alternatives are not included in AltPipe and will not be provided in the output Alternative pipe list: all non-circular shapes (arches, boxes, etc.), non reinforced concrete pipe (NRCP) and non-standard new products. Check Materials and Hydraulics reports and verify if any of these alternatives were recommended and supplement the AltPipe final summary accordingly. For reinforced concrete pipe (RCP), box (RCB) and arch (RCA) culverts, maintenance-free service life, with respect to corrosion, abrasion and/or durability, is the number of years from installation until the deterioration reaches the point of exposed reinforcement at any point on the culvert. Changes in the design may be required in relatively severe acidic, chloride or sulfate environments. The levels of these constituents (either in the soil or water) will need to be identified in the project Materials or Geotechnical Design Report. The adopted procedure consists of a formula that the constituent concentrations are entered into in order to determine a pipe service life. The means for offsetting the affects of the corrosive elements is to increase the cover over the reinforcing steel, increase the cement content, or reduce the water/cement ratio.

Step 7. Table 855.2C constitutes a guide for abrasive resistant coatings in low to moderate abrasive conditions for metal pipe (i.e., Levels 1 through 5 in Table 855.2A) and is included in AltPipe. Table 855.2F constitutes a guide for minimum material thickness of abrasive resistant invert protection to achieve 50 years of maintenance-free service life in moderate to highly abrasive conditions (i.e., Levels 4 through 6 in Table 855.2A) and was not programmed into AltPipe. If pipe material thickness does not meet service life due to abrasive conditions, consideration for invert protection should be made using Table 855.2F as a guide.

857.3 Alternative Pipe Culvert (APC) and Pipe Arch Culvert List

Because of the difference in roughness coefficients between various materials, it may be necessary to specify a different size for each allowable material at any one location. In this event, it is recommended that the material with the smallest dimension be listed as the alternative size. Refer to Plans Preparation Manual for standard format to be used.

There may be situations where there is a different set of alternatives for the same nominal size of alternative drainage facilities. In this case the different sets of the same nominal size should be further identified by different types, for example, 18-inch alternative pipe culvert (Type A), 18-inch alternative pipe culvert (Type B), etc. No attempt to correlate type designation between projects is necessary. The first alternative combination for each culvert size on each project should be designated as Type A, second as Type B, etc.

Since the available nominal sizes for pipe arches vary slightly between pipe arch materials, it is recommended that the listed alternative pipe arch sizes conform to those sizes shown for corrugated steel pipe arches shown on Table 856.3D. The designer should verify the availability of reinforced concrete pipe arches. If reinforced concrete pipe arches are not available, oval shaped reinforced concrete pipe of a size necessary to meet the hydraulic requirements may be used as an alternative.

CHAPTER 860
ROADSIDE CHANNELS

Topic 861 – General

Index 861.1 - Introduction

Chapter 860 addresses the design of small open channels called roadside channels that are constructed as part of a highway drainage system. See Figure 861.1.

Figure 861.1
Small Roadside Channel

An open channel is a conveyance in which water flows with a free surface. Although closed conduits such as culverts and storm drains function as open channels when flowing partially full, the term is generally applied to natural and improved watercourses, gutters, ditches, and channels. While the hydraulic principles discussed in this chapter are valid for all drainage structures, the primary consideration is given to roadside channels.

In addition to performing its hydraulic function, the roadside channel should be economical to construct and maintain. Some roadside channels serve as dual purpose channels which concurrently function as infiltration swales for stormwater purposes. See Index 861.11, "Water Quality Channels". Roadside channel design should consider errant vehicles leaving the traveled way, be pleasing in appearance, convey collected water without damage to the transportation facility or adjacent property and minimize environmental impacts. These considerations are usually so interrelated that optimum conditions cannot be met for one without compromising one or more of the others. The objective is to achieve a reasonable balance, but the importance of traveler safety must not be underrated. See Index 861.4, "Safety Considerations".

Roadside channels play an important role in the highway drainage system as the initial conveyance for highway runoff. Roadside channels are often included as part of the typical roadway section. Therefore, the geometry of roadside channels depends on available right-of-way, flow capacity requirements, and the alignment and profile of the highway. Most roadside channels capture sheet flow from the highway pavement and cut slope and convey that runoff to larger channels or to culverts within the drainage system. See Figure 861.2.

Figure 861.2
Roadside Channel Outlet to Storm Drain at Drop Inlet

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

861.2 Hydraulic Considerations

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

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

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

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

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

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

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

861.3 Selection of "Design Flood"

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

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

861.4 Safety Considerations

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

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

861.5 Maintenance Considerations

Design of open channels and roadside ditches should recognize that periodic maintenance inspection and repair is required. Provisions should be incorporated into the design for access to a channel by maintenance personnel and equipment. Consideration should be given to the size and type of maintenance equipment required when assessing the need for permanent or temporary access easements for entrance ramps and gates through the right of way fences.

Damaged channels can be expensive to repair and interfere with the safe and orderly movement of traffic.

Figure 861.3
Damaged Channel

Minor erosion damage within the right of way should be repaired immediately after it occurs and action taken to prevent the recurrence. Conditions which require extensive repair or frequently recurring maintenance may require a complete redesign rather than repetitive or extensive reconstruction. The advice of the District Hydraulics Engineer should be sought when evaluating the need for major restoration.

The growth of weeds, brush, and trees in a drainage channel can effectively reduce its hydraulic efficiency. See Figure 861.4. The result being that a portion of the design flow may overflow the channel banks causing flooding and possible erosion.

Figure 861.4
Concrete Lined Channel with Excessive Weed Growth

Accumulation of sediment and debris may destroy vegetative linings leading to additional erosion damage.

Channel work on some projects may be completed several months before total project completion. During this interim period, the contractor must provide interim protection measures. Per Index 865.3(3), the design engineer should include temporary channel linings to assure that minor erosion will not develop into major damage. As needed, the District Project Engineer may obtain vegetative recommendations from the District Landscape Architect. The Project Engineer must verify vegetative component compatibility with the final design.

861.6 Economics

Economical drainage design is achieved by selecting the design alternative which best satisfies the established design criteria at the lowest cost.

The economic evaluation of design alternatives should be commensurate with the complexity and importance of the facility. Analysis of the channel location, shape, size, and materials involved may reveal possibilities for reducing construction costs, flood damage potential, maintenance problems and environmental impacts.

861.7 Coordination with Other Agencies

There are many Federal, State and local agencies and private entities engaged in water related planning, construction and regulation activities whose interests can affect the design of some highway drainage channels (e.g., flood control channels described under Index 861.2(1)). Such agencies may request the channel design satisfy additional and perhaps governing design criteria. Early coordination with these agencies may help avoid delays in the project development process and post-project conflicts. Early coordination may also reveal opportunities for cooperative projects which may benefit both Caltrans and the water resources agency. For information on cooperative agreements refer to Index 803.2.

861.8 Environment

Many of the same principles involved in sound highway construction and maintenance of open channels parallel environmental considerations. Environmental problems can arise if riparian species inhabit the channel. Erosion, sedimentation, water quality, and aesthetics should be of prime concern to the highway design engineer. Refer to Index 110.2 and the Project Planning and Design Guide for discussion on control of water pollution.

861.9 Unlined Channels

Whenever feasible, roadside channels should be designed with natural bottoms. Use linings only when warranted.

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

861.10 Lined Channels

The main purposes of channel linings are:

(a) To prevent erosion damage.

(b) To increase velocity for prevention of excessive sedimentation

(c) To increase capacity.

See Topic 865 for design concepts.

861.11 Water Quality Channels

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

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

http://www.dot.ca.gov/hq/oppd/stormwtr/ppdg/swdr2012/PPDG-May-2012.pdf

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

http://www.dot.ca.gov/hq/oppd/storm1/caltrans_20090729.html

861.12 References

More complete information on hydraulic principles and engineering techniques of open channel design may be found in FHWA's Hydraulic Design Series No. 3, "Design Charts for Open Channel Flow", Hydraulic Design Series No. 4, "Introduction to Highway Hydraulics", Hydraulic Engineering Circular No. 15 (HEC No. 15), "Design of Roadway Channels with Flexible Linings" and Hydraulic Engineering Circular No. 22 (HEC No. 22), Chapter 5, "Urban Drainage Design Manual – Roadside and Median Channels". For a general textbook discussion of open channel hydraulics, reference is made to "Open-Channel Hydraulics" by Ven Te Chow. In addition, many helpful design aids are included in "Handbook of Hydraulics", by Brater and King.

Topic 862 - Roadside Drainage Channel Location

862.1 General

Assuming adequate functional design, the next most important design consideration is channel location. Locations that avoid poorly drained areas, unstable soil conditions, and frequently flooded areas can greatly reduce drainage related problems. Refer to Index 110.4 for discussion on wetlands protection.

Typically drainage and open channel considerations are not considered the primary decision factors in the roadway location; however they are factors which will often directly or indirectly affect many other considerations. Often minor alignment adjustments can avoid serious drainage problems.

If a channel can be located far enough away from the highway, the concerns of traffic safety and aesthetics can be significantly mitigated. See Figure 862.1. The cost of additional right of way may be offset somewhat by the reduced cost of erosion control, traffic protection, and landscaping.

Figure 862.1
Small-Rock Lined Channel Outside of Clear Recovery Zone

862.2 Alignment and Grade

Ordinarily, the highway drainage channel must be located where it will best serve its intended purpose, using the grade and alignment obtainable at the site. Insofar as practicable, abrupt changes in alignment and grade should be avoided. A sharp change in alignment presents a point of attack for flowing water, and abrupt changes in grade can result in possible scour when the grade is steepened or deposition of transported material when the grade is flattened.

Ideally, a drainage channel should have flow velocities that neither erode nor cause deposition in the channel. This optimum velocity is dependent on the size and slope of channel, the quantity of flowing water, the material used to line the channel, the nature of the bedding soil and the sediment being transported by the flow. Refer to Table 865.2 for recommended permissible flow velocities in unlined channels.

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

862.3 Point of Discharge

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

Topic 863 - Channel Section

863.1 Roadside and Median Channels

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

Reference is made to the FHWA publication HEC No. 22, Chapter 5.

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

863.2 Triangular

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

Figure 863.1
Small-Rock Lined Triangular Channel with Rounded Bottom

863.3 Trapezoidal

The most common channel shapes is the trapezoidal section.

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

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

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

where:

L = Length of vertical curve in feet

X = Horizontal component of side slopes expressed as x, y coordinates with y = 1

For narrow channels, L, is limited to the bottom width.

863.4 Rectangular

Rectangular channels are used to convey large flows in areas with limited right of way. At some locations, guardrail or other types of positive traffic barrier may be necessary between the traveled way and the channel.

Though rectangular channels are relatively expensive to construct, since the walls must be designed as earth retaining structures, the construction costs can be somewhat offset by the reduced costs associated with right of way, materials, and channel excavation. See Index 865.2 for the design of concrete lined flood control channels.

Topic 864 - Channel Stability Design Concepts

864.1 General

The gradient of roadside channels typically parallels the grade of the highway. Even at relatively mild highway grades, highly erosive hydraulic conditions can exist in adjacent roadside channels. Consequently, designing a stable conveyance becomes a critical component in the design of roadside channels.

The need for erosion prevention is not limited to the highway drainage channels; it extends throughout the right-of-way and is an essential feature of adequate drainage design. Erosion and maintenance are minimized largely by the use of flat side slopes rounded and blended with natural terrain, drainage channels designed with due regard to location, width, depth, slopes, alignment, and protective treatment, proper facilities for groundwater interception, dikes, berms, and other protective devices, and protective ground covers and planting.

864.2 Stable Channel Design Procedure

For most highway drainage channels bed and side slope instability cannot be tolerated and stable channel design must be based on the concepts of static equilibrium, including the use of a lining material if necessary. The permissible tractive force (shear stress) procedure requires that the shear stresses on the channel bottom and sides do not exceed the allowable amounts for the given channel boundary. Based on the actual physical processes involved in maintaining a stable channel, specifically the stresses developed at the interface between flowing water and materials forming the channel boundary, the tractive force procedure is a more realistic model and was adopted as the preferred design procedure for HEC No. 15, which is the primary reference for stable channel design.

The maximum shear stress along the channel bottom may be estimated by the following equation:

$$\tau_d = \gamma d S$$

where:

τ_d = Shear stress in channel at maximum depth, lb/ft^2

γ = Specific weight of water

d = Maximum depth of flow in channel for the design discharge, ft

S = Slope of channel, ft/ft

When the permissible shear stress is greater than or equal to the computed shear stress, the lining is considered acceptable:

$$\tau_p \geq SF\tau_d$$

where:

τ_p = Permissible shear stress for the channel lining, lb/ft

SF = Safety factor

The safety factor provides for a measure of uncertainty, as well as a means for the designer to reflect a lower tolerance for failure by choosing a higher safety factor. A safety factor of 1.0 is appropriate in many cases and may be considered the default. However, safety factors from 1.0 to 1.5 may be appropriate, subject to the designer's discretion, where one or more of the following conditions may exist:

(a) critical or supercritical flows are expected

(b) climatic regions where vegetation may be uneven or slow to establish

(c) significant uncertainty regarding the design discharge

(d) consequences of failure are high

The relationship between permissible shear stress and permissible velocity for a lining can be found by substituting the equation for maximum shear stress and continuity equation into Manning's equation:

$$V_p = \frac{\alpha}{n\sqrt{\gamma d}} R^{1/6} \tau_p^{1/2}$$

where:

V_p = Permissible velocity, ft/s

τ_p = Permissible shear stress, lb/ft^2

α = Unit conversion constant, 1.49

As a guide, Table 865.2 provides typical values of permissible velocity and permissible shear stress for selected lining types.

The basic procedure for designing a flexible lining consists of the following steps.

Step 1. Determine a design discharge, Q, and select the channel slope and channel shape.

Step 2. Select a trial lining type. Initially, the Engineer may need to determine if a long-term lining is needed and whether or not a temporary or transitional lining is required. For determining the latter, the trial lining type could be chosen as the native material (unlined), typically bare soil. For example, it may be determined that the bare soil is insufficient for a long-term solution, but vegetation is a good solution. For the transitional period between construction and vegetative establishment, analysis of the bare soil will determine if a temporary lining is prudent. Per Index 865.1, District Landscape should be consulted to provide feasible long-term vegetation recommendations. The Engineer and the Landscape Architect should discuss the compatibility of any living materials (temporary, transitional or permanent) with the proposed lining material and verify impacts to conveyance before the Engineer finalizes the design.

Step 3. Estimate the depth of flow, d_i in the channel and compute the hydraulic radius, R. The estimated depth may be based on physical limits of the channel, but this first estimate is essentially a guess. Iterations on Steps 3 through 5 may be required.

Step 4. Estimate Manning's n and the discharge implied by the estimated n and flow depth values. Calculate the discharge (Q_i).

Step 5. Compare Q_i with Q. If Q_i is within 5 percent of the design, Q, then proceed on to Step 6. If not, return to Step 3 and select a new estimated flow depth, d_{i+1}. This can be estimated from the following equation or any other appropriate method.

$$d_{i+1} = d_i \left(\frac{Q}{Q_i}\right)^{0.4}$$

Step 6. Calculate the shear stress at maximum depth, τ_d, determine the permissible shear stress, τ_p, according to the methods described in HEC No. 15 and select an appropriate safety factor (i.e., 1 to 1.5).

Step 7. Compare the permissible shear stress to the calculated shear stress from Step 6 using:

$$\tau_p \geq SF\tau_d$$

If the permissible shear stress is adequate then the lining is acceptable. If the permissible shear is inadequate, then return to Step 2 and select an alternative lining type with greater permissible shear stress from Table 865.2. As an alternative, a different channel shape may be selected that results in a lower depth of flow. The selected lining is stable and the design process is complete. Other linings may be tested, if desired, before specifying the preferred lining.

Direct solutions for Manning's equation for many channels of trapezoidal, rectangular, triangular and circular cross sections can be found within the Channel Analysis subcomponent FHWA's Hydraulic Toolbox software program.

864.3 Side Slope Stability

Shear stress is generally reduced on the channel sides compared with the channel bottom. The maximum shear on the side of a channel is given by the following equation:

$$\tau_s = K_1\tau_d$$

where:

τ_s = Side shear stress on the channel, lb/ft^2

K_1 = Ratio of channel side to bottom shear stress

τ_d = Shear stress in channel at maximum depth, lb/ft^2

The value K_1 depends on the size and shape of the channel. For parabolic or V-shape with rounded bottom channels there is no sharp discontinuity along the wetted perimeter and therefore it can be assumed that shear stress at any point on the side slope is related to the depth at that point using the shear stress equation from Index 864.2:

$$\tau_d = \gamma dS$$

For trapezoidal and triangular channels, the following K_1 values may be applied:

$K_1 = 0.77$ $\quad Z \leq 1.5$

$K_1 = 0.066Z + 0.67$ $\quad 1.5 < Z < 5$

$K_1 = 1.0$ $\quad 5 \leq Z$

The Z value represents the horizontal dimension 1:Z (V:H). Use of side slopes steeper than 1:3 (V:H) is not encouraged for flexible linings because of the potential for erosion of the side slopes. Steep side slopes are allowable within a channel if cohesive soil conditions exist. Channels with steep slopes should not be allowed if the channel is constructed in non-cohesive soils.

For channels lined with gravel or small-rock slope protection, the maximum suggested side slope is 1 V : 3 H, and flatter slopes are encouraged. If steeper side slopes are required, see Chapter 6 of HEC No. 15 for design procedures.

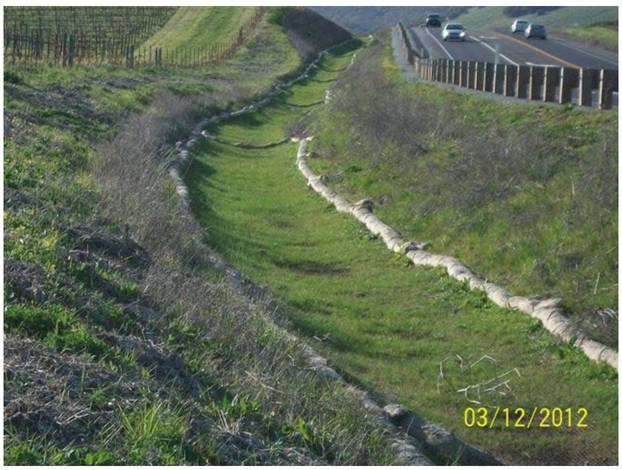
Topic 865 - Channel Linings

865.1 Flexible Verses Rigid

Lining materials may be classified as flexible or rigid. Flexible linings are able to conform to changes in channel shape and can sustain such changes while maintaining the overall integrity of the channel. In contrast, rigid linings cannot change shape and tend to fail when a portion of the channel lining is damaged. Channel shape may change due to frost-heave, slumping, piping, etc. Typical flexible lining materials include grass or small-rock slope protection, while typical rigid lining materials include hot mixed asphalt or Portland cement concrete. Flexible linings are generally less expensive, may have a more natural appearance, permit infiltration and exfiltration and are typically more environmentally acceptable. Vegetative channel lining is also recognized as a best management practice for storm water quality design in highway drainage systems. A vegetated channel helps to deposit highway runoff contaminants (particularly suspended sediments) before they leave the highway right of way and enter streams. See Index 861.11 ‘Water Quality Channels’ and Figure 865.1.

On steep slopes, most vegetated flexible linings are limited in the erosive forces they can sustain without damage to the channel and lining unless the vegetative lining is combined with another more erosion-resistant long-term lining below, such as a cellular soil confinement system. See Figure 865.1 and Index 865.3(1). The District Landscape Architect should be contacted to provide viable vegetation alternatives within the District, however all design responsibilities belong to the Project Engineer.

Figure 865.1
Steep-Sloped Channel with Composite Vegetative Lining



Vegetative flexible lining placed on top of cellular soil confinement system on a steep-sloped channel.

865.2 Rigid

A rigid lining can typically provide higher capacity and greater erosion resistance and in some cases may be the only feasible alternative.

Rigid linings are useful in flow zones where high shear stress or non-uniform flow conditions exist, such as at transitions in channel shape or at an energy dissipation structure.

The most commonly used types of rigid lining are hot mixed asphalt and Portland cement concrete. Hot mixed asphalt is used mainly for small ditches, gutters and overside drains (see Standard Plan D87D) because it cannot withstand hydrostatic pressure from the outside.

Table 865.1 provides a guide for Portland cement concrete and air blown mortar roadside channel linings. See photo below Table 865.1 for example.

For the design of concrete lined flood control channels discussed in Index 861.2 (1), see U.S. Army Corps of Engineers publication; "Structural Design of Concrete Lined Flood Control Channels", EM 1110-2-2007:

http://planning.usace.army.mil/toolbox/library/EMs/em1110.2.2007.pdf

Table 865.1
Concrete(2) Channel Linings

Abrasion Level(1)	Thickness of Lining (in)		Minimum Reinforcement
	Sides	Bottom	
1 - 3	5	5	6 x 6-W2.9 x W2.9 welded wire fabric

NOTES:

(1) See Table 855.2A.

(2) Portland Cement Concrete or Air Blown Mortar

Figure 865.2
Concrete Lined Channel

For large flows, consideration should be given to using a minimum bottom width of 12 feet for construction and maintenance purposes, but depths of flow less than one foot are not recommended.

Despite the non-erodible nature of rigid linings, they are susceptible to failure from foundation instability and abrasion. The major cause of failure is undermining that can occur in a number of ways.

865.3 Flexible

Flexible linings can be long-term, transitional or temporary. Long-term flexible linings are used where the channel requires protection against erosion for the design service life of the channel. Per Index 861.12, more complete information on hydraulic principles and engineering techniques of flexible channel lining design may be found in HEC No. 15 and Chapter 5 of HEC No. 22.

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

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

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

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

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

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

Figure 865.3
Long-Term Flexible Lining

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

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

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

http://www.dot.ca.gov/hq/LandArch/ec/recp/trm.htm

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

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

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

865.4 Composite Lining Design

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

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

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

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

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

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

where:

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

P_L = Low flow lining perimeter, ft

P = Total flow perimeter, ft

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

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

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

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

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

865.5 Bare Soil Design and Grass Lining

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

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

Figure 865.4
Grass-Lined Median Channel

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

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

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

where:

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

α = Unit conversion constant, 0.213

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

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

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

where:

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

τ_d = Design shear stress, lb/ft^2

C_f = Grass cover factor (use 0.6 to 0.8 or Table 4.5 from HEC-15)

n_s = Soil grain roughness

n = Overall lining roughness

The soil grain roughness, n_s, is 0.016 when D_{75} < 0.05 in. For larger grained soils the soil grain roughness is

$$n_s = \alpha (D_{75})^{1/6}$$

where:

n_s = Soil grain roughness (D_{75} > 1.3 (0.05 in))

D_{75} = Soil size where 75 percent of the material is finer, in

α = Unit conversion constant, 0.026

The permissible soil shear stress for fine-grained, non-cohesive soils (D_{75} < 0.05 in. is relatively constant and is conservatively estimated at 0.02 lb/ft^2. For coarse grained, non-cohesive soils (0.05 in. < D_{75} < 2 in.) the following equation applies.

$$\tau_{p,soil} = \alpha D_{75}$$

where:

$\tau_{p,soil}$ = Permissible soil shear stress, lb/ft^2

D_{75} = Soil size where 75 percent of the material is finer, in

α = Unit conversion constant, 0.4

Table 865.2[2]
Permissible Shear and Velocity for Selected Lining Materials

Boundary Category	Boundary Type	Permissible Shear Stress (lb/ft^2)	Permissible Velocity (ft/s)
Soils[1]	Fine colloidal sand	0.03	1.5
	Sandy loam (noncolloidal)	0.04	1.75
	Clayey sands (cohesive, PI ≥ 10)	0.095	2.6
	Inorganic silts (cohesive, PI ≥ 10)	0.11	2.7
	Silty Sands (cohesive, PI ≥ 10)	0.072	2.4
	Alluvial silt (noncolloidal)	0.05	2
	Silty loam (noncolloidal)	0.05	2.25
	Finer than course sand - D_{75} < 0.05 in. (non-cohesive)	0.02	1.3
	Firm loam	0.075	2.5
	Fine gravels	0.075	2.5
	Fine gravel (non-cohesive, D_{75} = 0.3 in, PI<10)	0.12	2.8
	Gravel (D_{75} = 0.6 in) (non-cohesive, D_{75} = 0.6 in, PI<10)	0.24	3.7
	Inorganic clays (cohesive, PI ≥ 20)	0.14	2.9
	Stiff clay	0.25	4.5
	Alluvial silt (colloidal)	0.25	3.75
	Graded loam to cobbles	0.38	3.75
	Graded silts to cobbles	0.43	4
	Shales and hardpan	0.67	6
Vegetation	Class A turf (Table 4.1, HEC No. 15)	3.7	8
	Class B turf (Table 4.1, HEC No. 15)	2.1	7
	Class C turf (Table 4.1, HEC No. 15)	1.0	3.5
	Long native grasses	1.7	6
	Short native and bunch grass	0.95	4

Table 865.2[2] (con't.)
Permissible Shear and Velocity for Selected Lining Materials

Boundary Category	Boundary Type	Permissible Shear Stress (lb/ft^2)	Permissible Velocity (ft/s)
Rolled Erosion Control Products (RECPs)			
Temporary Degradable Erosion Control Blankets (ECBs)	Single net straw	1.65	3
	Double net coconut/straw blend	1.75	6
	Double net shredded wood	1.75	6
Open Weave Textile (OWT)	Jute	0.45	2.5
	Coconut fiber	2.25	4
	Vegetated coconut fiber	8	9.5
	Straw with net	1.65	3
Non Degradable Turf Reinforcement Mats (TRMs)	Unvegetated	3	7
	Partially established	6.0	12
	Fully vegetated	8.00	12
Rock Slope Protection, Cellular Confinement and Concrete			
Rock Slope Protection	Small-Rock Slope Protection (4-inch Thick Layer)	0.8	6
	Small-Rock Slope Protection (7-inch Thick Layer)	2	8
	No. 2	2.5	10
	Facing	5	12
Gabions	Gabions	6.3	12
Cellular Confinement: Vegetated infill	71 in^2 cell and TRM	11.6	12
Cellular Confinement: Aggregate Infill	1.14 - in. D_{50} (45 in^2 cell)	6.9	12
	3.5" D_{50} (45 in^2 cell)	15.1	11.5
	1.14" D_{50} (71 in^2 cell)	13.2	12
	3.5" D_{50} (71 in^2 cell)	18	11.7
	1.14" D_{50} (187 in^2 cell)	10.92	12
	3.5" D_{50} (187 in^2 cell)	10.55	12
Cellular Confinement: Concrete Infill	(71 in^2 cell)	2	12
Hard Surfacing	Concrete	12.5	12

NOTES:

(1) PI = Plasticity Index (From Materials or Geotechnical Design Report)

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

A simplified approach for estimating the permissible shear stress for cohesive soils (based on Equation 4.6 in Chapter 4 of HEC No. 15) is illustrated in Figure 4.1 of Chapter 4 in HEC No. 15. The combined effects of the soil permissible shear stress and the effective shear stress transferred through the vegetative lining results in a permissible shear stress for the given conditions. Table 865.2 provides typical values of permissible shear stress and permissible velocity for cohesive soils and selected lining types. Representative values for different soil, vegetation and lining types are based on the methods found in Chapter 4 of HEC No. 15 while those for gravel, rock gabions and rock slope protection are based on methods found in Chapters 6 and 7 of HEC No. 15. The permissive shear stress values shown for soil confinement systems are based on testing by others, however, the maximum permissive velocity shown in Table 865.2 for all boundary types has been limited to 12 feet per second based on the following assumptions:

- The upper limit of flow rate is 50 cfs
- The longitudinal slope is 10 percent maximum
- The maximum side slope is 2H:1V
- The maximum storm duration is one hour

When the permissible shear stress is greater than or equal to the computed shear stress, the lining is considered acceptable. If the computed velocity exceeds the permissive velocity, or any of the above-listed assumptions are exceeded, contact the District Hydraulic Engineer for support.

865.6 Rolled Erosion Control Products

(1) *General.* Manufacturers have developed a variety of rolled erosion control products (RECPs) for erosion protection of channels.

RECPs consist of materials that are stitched or bound into a fabric. Vegetative and RECP lining performance relates to how well they protect the underlying soil from shear stresses so these linings do not have permissible shear stresses independent of soil types. Chapters 4 (vegetation) and 5 (RECPs) of HEC No. 15 describe the methods for analyzing these linings. Standard Specification Section 21-1 was developed primarily to address slope erosion products, however, the specifications for constructing turf reinforcing mats (TRM's), open weave textiles and erosion control blankets may also be applied to channels as temporary and transitional linings, and some TRM's may be used as permanent linings.

(2) *Non-Hydraulic Design Considerations.* The long-term performance of TRMs has traditionally been evaluated using hydraulic testing performance within controlled flume environments, or laboratory testing of specific parameters, usually conforming to ASTM or other industry standards. In recent years additional important design factors have been identified, from damages due to insect infestation to drainage problems or soil conditions resulting in poor vegetative establishment. Table 5.5 within Chapter 5 of HEC No. 15 provides a detailed TRM protocol checklist.

Six broad categories of stressors or potential damages to RECPs are listed below that can cause decrease in performance, considered as a function of specific properties of these lining materials.

(a) Environmental stress – tensile stresses that exceed the mechanical strength of the material accelerated by other stresses in the exposure environment.

Many manufacturer-reported values for maximum velocity or shear stress are based on short duration testing, however, longer duration flows – hours to days – more closely represent field conditions. Erosive properties of soils change with saturation, vegetation becomes stressed or damaged, and properties of some lining materials change with long periods of inundation or hydraulic stress. The result is that maximum reported shear stress and velocity may overestimate actual field performance of the full range of channel lining materials in the event of longer duration flows (Table 865.2). See Index 865.5 for safety factor discussion.

(b) Mechanical damage – localized damage due to externally applied loads such as debris or machinery, often during installation but also due to operation and maintenance activities

(c) Oxidation – due to exposure to air and water, a chemical reaction with a specific chemical group in a constituent polymer that leads to damage at a molecular level and changes in physical properties. Other chemical stresses can include acidity, corrosives, salinity, ozone and other air pollutants.

(d) Photo degradation – change in chemical structure due to exposure to UV wavelengths of sunlight, most often occurring during installation, prior to full vegetation establishment or inadequate vegetation establishment and coverage over time.

UV-Resistance per ASTM D-4355 should conform to the following for the specified type of TRM and design life:

- Temporary or transitional TRM – 90% tensile strength retained at 500 hr for the TRM product to be considered up to a 5-year design life.
- Long-term TRM – 90% tensile strength retained at 5,000 hr for the TRM product to be considered up to a 50-year design life.

(e) Temperature instability – changes in appearance, weight, dimension or other properties as a result of low, high, or cyclic temperature exposure.

As TRM or other materials are degrading, the vegetative component of a project is simultaneously becoming established, presumably leading to an overlap in effectiveness of each component. The engineer must carefully evaluate published performance data for specific materials with anticipated degradation, consider specific performance added by vegetative components, and apply a factor of safety in choosing materials that may provide enough strength initially to bridge the gap. Per Index 865.6(1), the District Landscape Architect should be consulted to provide viable long-term and compatible transitional vegetation recommendations (if required by the designer).

Topic 866 - Hydraulic Design of Roadside Channels

866.1 General

Open channel hydraulic design is of particular importance to highway design because of the interrelationship of channels to most highway drainage facilities.

The hydraulic principles of open channel flow are based on steady state uniform flow conditions, as defined in Index 866.2. Though these conditions are rarely achieved in the field, generally the variation in channel properties is sufficiently small that the use of uniform flow theory will yield sufficiently accurate results for most roadside channels.

866.2 Flow Classifications

(1) Steady vs. Unsteady Flow. The flow in an open channel can be classified as steady or unsteady. The flow is said to be steady if the depth of flow at a section, for a given discharge, is constant with respect to time. The flow is considered unsteady if the depth of flow varies with respect to time.

(2) Uniform Flow. Steady flow can further be classified as uniform or nonuniform. The flow is said to be uniform if the depth of flow and quantity of water are constant at every section of the channel under consideration. Uniform flow can be maintained only when the shape, size, roughness and slope of the channel are constant. Under uniform flow conditions, the depth and mean velocity of flow is said to be normal. Under these conditions the water surface and flowlines will be parallel to the stream bed and a hydrostatic pressure condition will exist, the pressure at a given section will vary linearly with depth.

As previously mentioned, uniform flow conditions are rarely attained in the field, but the error in assuming uniform flow in a channel of fairly constant slope, roughness and cross section is relatively small when compared to the uncertainties of estimating the design discharge.

(3) *Non-uniform Flow.* There are two types of steady state non-uniform flow:

- Gradually varied flow.

  Gradually varied flow is described as a steady state flow condition where the depth of water varies gradually over the length of the channel. Under this condition, the streamlines of flow are practically parallel and therefore, the assumption of hydrostatic pressure distribution is valid and uniform flow principles can be used to analyze the flow conditions.

- Rapidly varied flow.

  With the rapidly varied flow condition, there is a pronounced curvature of the flow streamlines and the assumption of hydrostatic pressure distribution is no longer valid, even for the continuous flow profile. A number of empirical procedures have been developed to address the various phenomena of rapidly varied flow. For additional discussion on the topic of rapidly varied flow, refer to "Open-Channel Hydraulics" by Chow.

866.3 Open Channel Flow Equations

The equations of open channel flow are based on uniform flow conditions. Some of these equations have been derived using basic conservation laws (e.g. conservation of energy) whereas others have been derived using an empirical approach.

(1) *Continuity Equation.* One of the fundamental concepts which must be satisfied in all flow problems is the continuity of flow. The continuity equation states that the mass of fluid per unit time passing every section in a stream of fluid is constant. The continuity equation may be expressed as follows:

$$Q = A_1V_1 = A_2V_2 = \ldots = A_nV_n$$

Where Q is the discharge, A is the cross-sectional flow area, and V is the mean flow velocity. This equation is not valid for spatially varied flow, i.e., where flow is entering or leaving along the length of channel under consideration.

(2) *Bernoulli Equation.* Water flowing in an open channel possesses two kinds of energy: (1) potential energy and (2) kinetic energy. Potential energy is due to the position of the water surface above some datum. Kinetic energy is due to the energy of the moving water. The total energy at a given section as expressed by the Bernoulli equation is equal to:

$$H = z + d + \frac{V^2}{2g}$$

where:

H = Total head, in feet of water

z = Distance above some datum, in feet

d = Depth of flow, in feet

$\frac{V^2}{2g}$ = Velocity head, in feet

g = Acceleration of gravity

= 32.2 feet per second squared

(3) *Energy Equation.* The basic principle used most often in hydraulic analysis is conservation of energy or the energy equation. For uniform flow conditions, the energy equation states that the energy at one section of a channel is equal to the energy at any downstream section plus the intervening energy losses. The energy equation, expressed in terms of the Bernoulli equation, is:

$$z_1 + d_1 + \frac{V_1^2}{2g} = z_2 + d_2 + \frac{V_2^2}{2g} + h_L$$

where:

h_L = Intervening head losses, in feet

(4) *Manning's Equation.* Several equations have been empirically derived for computing the average flow velocity within an open channel. One such equation is the Manning Equation. Assuming uniform and turbulent flow conditions, the mean flow velocity in an open channel can be computed as:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

where:

V = Mean velocity, in feet per second

n = Manning coefficient of roughness

S = Channel slope, in foot per feet

R = Hydraulic Radius, in feet

= A/WP

where:

A = Cross sectional flow area, in square feet

WP = Wetted perimeter, in feet

Commonly accepted values for Manning's roughness coefficient, n, based on materials and workmanship required in the Standard Specifications, are provided in Table 866.3A. The tabulated values take into account deterioration of the channel lining surface, distortion of the grade line due to unequal settlement, construction joints and normal surface irregularities. These average values should be modified to satisfy any foreseeable abnormal conditions. See Chapters 4 and 6 in HEC No. 15 for Manning's roughness equations for grass linings, RSP, cobble and gravel linings. Refer to Index 861.11 for a discussion of Manning's roughness coefficients for water quality channels.

Direct solutions for Manning's equation for many channels of trapezoidal, rectangular, triangular and circular cross sections can be found within the Channel Analysis subcomponent FHWA's Hydraulic Toolbox software program.

(5) *Conveyance Equation.* Often it is convenient to group the properties peculiar to the cross section into one term called the conveyance factor, K. The conveyance factor, as expressed by the Manning's equation, is equal to:

$$K = \frac{1.486}{n} AR^{2/3}$$

For the non-pressure, full flow condition, the geometric properties and conveyance of a channel section can be computed. Then for a given channel slope the discharge capacity can be easily determined.

(6) *Critical Flow.* A useful concept in hydraulic analysis is that of "specific energy". The specific energy at a given section is defined as the total energy, or total head, of the flowing

Table 866.3A

Average Values for Manning's Roughness Coefficient (n)

Type of Channel	n value
Unlined Channels:	
Clay Loam	0.023
Sand	0.020
Gravel	0.030
Rock	0.040
Lined Channels:	
Portland Cement Concrete	0.014
Air Blown Mortar (troweled)	0.012
Air Blown Mortar (untroweled)	0.016
Air Blown Mortar (roughened)	0.025
Asphalt Concrete	0.016-0.018
Sacked Concrete	0.025
Pavement and Gutters:	
Portland Cement Concrete	0.013-0.015
Hot Mix Asphalt Concrete	0.016-0.018
Depressed Medians:	
Earth (without growth)	0.016 - 0.025
Earth (with growth)	0.050
Gravel (d_{50} = 1 in. flow depth $\leq$ 6 in.)	0.040
Gravel (d_{50} = 2 in. flow depth $\leq$ 6 in.)	0.056

NOTES:

For additional values of n, see HEC No. 15, Tables 2.1 and 2.2, and "Introduction to Highway Hydraulics", Hydraulic Design Series No. 4, FHWA Table 14.

water with respect to the channel bottom. For a channel of small slope;

$$E = d + \frac{V^2}{2g}$$

where:

E = Specific energy, in feet

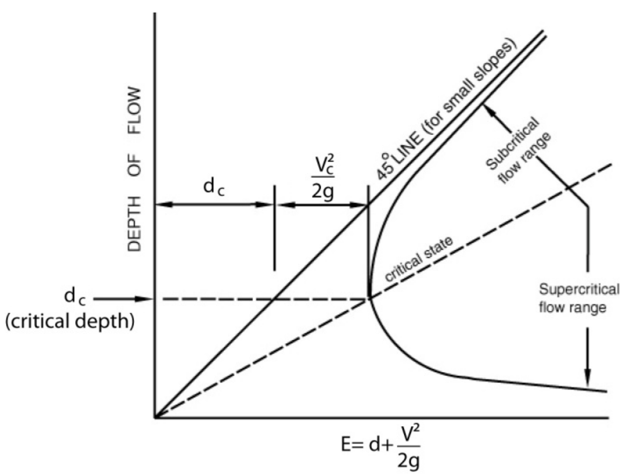
d = Depth of flow, in feet

$\frac{V^2}{2g}$ = Velocity head, in feet

When the depth of flow is plotted against the specific energy, for a given discharge and channel section, the resulting plot is called a specific energy diagram (see Figure 866.3C). The curve shows that for a given specific energy there are two possible depths, a high stage and a low stage. These flow depths are called alternate depths. Starting at the upper right of the curve with a large depth and small velocity, the specific energy decreases with a decrease in depth, reaching a minimum energy content at a depth of flow known as critical depth. A further decrease in flow depth results in a rapid increase in specific energy.

Flow at critical depth is called critical flow. The flow velocity at critical depth is called critical velocity. The channel slope which produces critical depth and critical velocity for a given discharge is the critical slope.

Figure 866.3C
Specific Energy Diagram



Uniform flow within approximately 10 percent of critical depth is unstable and should be avoided in design, if possible. The reason for this can be seen by referring to the specific energy diagram. As the flow approaches critical depth from either limb of the curve, a very small change in energy is required for the depth to abruptly change to the alternate depth on the opposite limb of the specific energy curve. If the unstable flow region cannot be avoided in design, the least favorable type of flow should be assumed for the design.

When the depth of flow is greater than critical depth, the velocity of flow is less than critical velocity for a given discharge and hence, the flow is subcritical. Conversely, when the depth of flow is less than critical depth, the flow is supercritical.

When velocities are supercritical, air entrainment may occur. This produces a bulking effect which increases the depth of flow. For concrete lined channels, the normal depth of flow with bulking can be computed by using a Manning's "n" value of 0.018 instead of the 0.014 value given in Table 866.3A. Air entrainment also causes a reduction in channel friction with a resulting increase in flow velocity. A Manning's "n" value of about 0.008 is recommended for computing the velocity and specific energy of flow in concrete-lined channels carrying supercritical flow

Critical depth is an important hydraulic parameter because it is always a hydraulic control. Hydraulic controls are points along the channel where the water level or depth of flow is limited to a predetermined level or can be computed directly from the quantity of flow. Flow must pass through critical depth in going from subcritical flow to supercritical flow. Typical locations of critical depth are at:

(a) Abrupt changes in channel slope when a flat (subcritical) slope is sharply increased to a steep (supercritical) slope,

(b) A channel constriction such as a culvert entrance under some conditions,

(c) The unsubmerged outlet of a culvert on subcritical slope, discharging into a wide channel or with a free fall at the outlet, and

(d) The crest of an overflow dam or weir.

Critical depth for a given channel is dependent on the channel geometry and discharge only, and is independent of channel slope and roughness.

When flow occurs at critical depth the following relationship must be satisfied

$$\frac{A^3}{T} = \frac{Q^2}{g}$$

where:

A = Cross sectional area, ft^2

T = Top width of water surface, ft

Q = Discharge, CFS

g = Acceleration of gravity, 32.2 ft/s^2

Critical depth formulas, based on the above equation, for various channel cross-sections include:

- Rectangular sections,

$$d_c = \left(\frac{q^2}{g}\right)^{1/3}$$

Where:

q = Flow per unit width, CFS

- Trapezoidal sections. The tables in King's "Handbook of Hydraulics" provide easy solutions for critical depth for channels of varying side slopes and bottom widths.
- Circular sections. The tables in King's "Handbook of Hydraulics" can be used for obtaining easy solutions for critical depth.

(7) *Froude Number*. The Froude number is a useful parameter which uniquely describes open flow. The Froude number is a dimensionless value:

$$Fr = \frac{V}{(gD)^{1/2}}$$

Where:

D = A/T = Hydraulic depth, in feet

Fr < 1.0 ==> Subcritical flow

Fr = 1.0 ==> Critical flow

Fr > 1.0 ==> Supercritical flow

866.4 Water Surface Profiles

Depending on the site conditions, accuracy required, and risks involved, a single section analysis may be sufficient to adequately describe the channel stage discharge relationship. The basic assumptions to a single section analysis are uniform cross section, slope, and Manning's "n" values which are generally applicable to most roadside and median channels. The condition of uniform flow in a channel at a known discharge is computed using the Manning's equation combined with the continuity equation:

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2}$$

The depth of uniform flow is solved by rearranging Manning's Equation to the form the given below. This equation is solved by trial and error by varying the depth of flow until the left side of the equation is zero:

$$\frac{Qn}{1.49S^{1/2}} - AR^{2/3} = 0$$

Per Index 866.3 (4), direct solutions for Manning's equation for many channels of trapezoidal, rectangular, triangular and circular cross sections can be found within the Channel Analysis subcomponent FHWA's Hydraulic Toolbox software program.

Where uniform flow conditions do not adequately describe the actual flow conditions (e.g., natural channels) or where additional accuracy is desired, the computation of complete water surface profiles for each discharge value may be necessary using detailed backwater analysis methods. Per Index 802.1(4)(g) contact the District Hydraulic Engineer for support.

Topic 867 - Channel Changes

867.1 General

Chapter 860 primarily addresses the design of small man-made open channels called roadside channels (gutters, ditches, swales etc.) that are constructed as part of a highway drainage system. However, both the terms ‘open channel’ or ‘channel’ may be applied to any natural or improved watercourse as well as roadside channels. See Index 861.1.

A channel change is any realignment or change in the hydraulic characteristics of an existing channel. Per Index 802.1(4)(g), contact the District Hydraulic Engineer for support.

The main reasons for channel changes to either natural or improved watercourses (flood control channels, irrigation channels etc.) within the right of way are to:

- Permit better drainage
- Improve flow conditions
- Protect the highway from flood damage
- Reduce right of way requirements

The guidelines in Topic 823 (Culvert Location) generally recommend alignment of the thalweg of the stream with the centerline of the culvert, however, for economic reasons, small skews should be eliminated, moderate skews retained and large skews reduced. Road crossings requiring fish passage are strongly encouraged to retain the natural alignment of the stream, regardless of the skew. Alignment of the culvert centerline with the channel approach angle aids debris passage during storm flows and minimizes hydraulic turbulence which may impede fish passage.

Sometimes a channel change may be to its vertical alignment. For example, inverted siphons or sag culverts may be used to carry irrigation channels crossing the right of way via vertical realignment entirely below the hydraulic grade line. However, maintenance concerns include sediment build-up and potential leakage problems with full-flow barrel(s). See Index 829.7(2) and Index 867.2 below.

867.2 Design Considerations

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

Channel changes may result in a decreased surface roughness or increased channel slope. As a result the following may occur:

- Higher velocities which result in damage due to scour
- Sedimentation and meandering at downstream end of channel change
- A flattened downstream gradient which progresses upstream undercutting the channel banks or highway fill
- Flattened downstream gradient or channel restrictions may create undesirable backwater conditions.

A channel change perched above the bottom of an old flood stage stream bed may cause the stream to return to its old channel during a subsequent flood. In addition, the designer should consult with Geotechnical Services to ensure that infiltration through the bank would not be problematic.

Topic 868 - Freeboard Considerations

868.1 General

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

866.2 Height of Freeboard

(1) *Straight Alignment*. In channels where overflow may cause substantial damage, a guide for freeboard height for channels on a straight alignment, is provided in Table 868.2

Table 868.2

Guide to Freeboard Height

Shape of Channel	Subcritical Flow	Supercritical Flow
Rectangular	0.1 He	0.20 d
Trapezoidal	0.2 He	0.25 d

where:

He = Energy head, in feet

d = Depth of flow, in feet for a straight alignment

(2) *Critical Flow*. An unstable zone of flow occurs where the flow is near critical state. This is characterized by random waves. An allowance for waves should be added to the normal depth when the slope of the channel is between 0.7 S_c and 1.3 S_c.

$$H_w = 0.25 d_c \left[1 - 11.1 \left(\frac{S}{S_c} - 1\right)^2\right]$$

where:

H_w = height of wave, in feet

d_c = critical depth, in feet

S = slope of channel, in foot per feet

S_c = critical slope, in foot per feet

(3) *Superelevation.* The height of freeboard discussed above does not provide for superelevation of the water surface on curved alignments.

Flow around a curve will result in a rise of the water surface on the outside of the curve and extra lining is necessary to guard against overtopping.

Additional freeboard is necessary in bends and can be calculated use the following equation:

$$\Delta d = \frac{V^2 T}{g R_c}$$

where:

Δd = Additional freeboard required because of superelevation, feet

V = Average channel veloctiy, ft/s

T = Water surface top width, ft

G = Acceleration due to gravity, ft/s^2

R_c = Radius of curvature of the bend to the channel centerline, ft

See HEC No. 15, Chapter 3, for shear stress considerations around bends.

CHAPTER 870 CHANNEL AND SHORE PROTECTION - EROSION CONTROL

Topic 871 - General

Index 871.1 - Introduction

Highways, bikeways, pedestrian facilities and appurtenant installations are often attracted to parallel locations along streams, coastal zones and lake shores. These locations are under attack from the action of waves and flowing water, and may require protective measures.

Channel and shore protection can be a major element in the design, construction, and maintenance of highways. This section deals with procedures, methods, devices, and materials commonly used to mitigate the damaging effects of flowing water and wave action on transportation facilities and adjacent properties. Potential sites for such measures should be reviewed in conjunction with other features of the project such as long and short term protection of downstream water quality, aesthetic compatibility with surrounding environment, and ability of the newly created ecological system to survive with minimal maintenance. See Index 110.2 for further information on water quality and environmental concerns related to erosion control.

Refer to Topic 870 for definitions of drainage terms.

871.2 Design Philosophy

In each district there should be a designer or advisor, usually the District Hydraulic Engineer, knowledgeable in the application of bank protection principles and the performance of existing works. Information is also available from headquarters specialists in the Division of Design and Structures Design in the Division of Engineering Services (DES). The most effective designs result from involvement with Design, Landscape Architecture, Structures, Construction, and Maintenance (for further discussion on functional responsibilities see Topic 802).

There are a number of ways to deal with the problem of wave action and stream flow.

- The simplest way and generally the surest of success and permanence, is to locate the facility away from the erosive forces. This is not always feasible or economical, but should be the first consideration. Locating the facility to higher ground or solid support should never be overlooked, even when it requires excavation of solid rock, since excavated rock may serve as a valuable material for protection at other points of attack.

- The most commonly used method is to armor the embankment with a more resistant material like rock slope protection. The type of material to be used for the protection is discussed under Topic 872.

- A third method is to reduce the force of the attacking water. This is often done by means of retards, permeable jetties and various plantings such as willows. Plantings once established not only reduce stream velocity near the bank during heavy flows, but their roots add structure to the bank material.

- Another method is to direct the attacking water away from the embankment. In the case of wave attack, additional beach may be created between the embankment and the water by means of groins and sills which trap littoral drift or hold imported sand. In the case of stream attack, a new channel can be created or the stream can be diverted away from the embankment by the use of jetties, baffles, deflectors, groins or spurs.

Combinations of the above four methods may be used. Even protective works destroyed in floods have proven to be effective and cost efficient in minimizing damage to transportation facilities.

Design of protective features should be governed by the importance of the facility and appropriate design principles. Some of the factors which should be considered are:

- *Roughness*. Revetments generally are less resistant to flow than the natural channel bank. Channel roughness can be significantly reduced if a rocky vegetated bank is denuded of trees and rock outcrops. When a rough natural bank is replaced by a smooth revetment, the current is accelerated, increasing its power to erode, especially along the toe and downstream end of the revetment. Except in narrowed channels, protective elements should approximate natural roughness. Retards, baffles and jetties can simulate the effect of trees and boulders along natural banks and in overflow channels.

- *Undercutting*. Particular attention must be paid to protecting the toe of revetments against undercutting caused by the accelerated current along smoothed banks, since this is the most common cause of bank failure.
- *Standardization*. Standardization should be a guide but not a restriction in designing the elements and connections of protective structures.
- *Expendability*. The primary objective of the design is the security of the transportation facility, not security of the protective structure. Less costly replaceable protection may be more economical than expensive permanent structures.
- *Dependability*. An expensive structure is warranted primarily where transportation facilities carry high traffic volumes, where no reasonable detour is available, or where facility replacement is very expensive.
- *Longevity*. Short-lived structures or materials may be economical for temporary situations. Expensive revetments should not be placed on banks likely to be buried in widened embankments, nor on banks attacked by transient meander of mature streams.
- *Materials*. Optimum use should be made of local materials, considering the cost of special handling. Specific gravity of stone is a major factor in shore protection and the specified minimum should not be lowered without increasing the mass of stones. For example, 10 percent decrease in specific gravity requires a 55 percent increase in mass (say from a 9 ton stone to a 14 ton stone) for equivalent protection.
- *Selection*. Selection of class and type of protection should be guided by the intended function of the installation.
- *Limits*. Horizontal and vertical limits of protection should be carefully designed. The bottom limit should be secure against toe scour. The top limit should not arbitrarily be at high-water mark, but above it if overtopping would cause excessive damage and below it if floods move slowly along the upper bank. The end limits should reach and conform to durable natural features or be secure with respect to design parameters.

871.3 Selected References

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

(a) FHWA Hydraulic Engineering Circulars (HEC) -- The following five circulars were developed to assist the designer in using various types of slope protection and channel linings:

- HEC 11, Design of Riprap Revetment (2000)
- HEC 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (2006)
- HEC 15, Design of Roadside Channels with Flexible Linings (2005).
- HEC 18, Evaluating Scour at Bridges (2001)
- HEC 20, Stream Stability at Highway Structures (2001)
- HEC 23, Bridge Scour and Stream Instability Countermeasures (2009)
- HEC 25, Highways in the Coastal Environment (2008)

(b) FHWA Hydraulic Design Series (HDS) No. 6, River Engineering for Highway Encroachments (2001) -- A comprehensive treatise of natural and man-made impacts and responses on the river environment, sediment transport, bed and bank stabilization, and countermeasures.

(c) AASHTO Highway Drainage Guidelines -- General guidelines for good erosion control practices are covered in Volume III - Erosion and Sediment Control in Highway Construction, and Volume XI - Guidelines for Highways Along Coastal Zones and Lakeshores.

(d) AASHTO Drainage Manual (MDM) (2003) – Refer to Chapters; 11 – Energy Dissipators; 16 – Erosion and Sediment Control; 17 – Bank Protection; and 18 – Coastal Zone. The MDM provides guidance on engineering practice in conformance with FHWA's HEC and HDS publications and other nationally recognized engineering policy and procedural documents.

(e) U.S. Army Corps of Engineers Manuals. The following manuals are used throughout the U.S. as a primary resource for the design and analysis of coastal features:

- Shore Protection Manual (SPM) (1984) – Comprehensive two volume guidance on wave and shore processes and methods for shore protection. No longer in publication but still referenced pending completion of the Coastal Engineering Manual.
- Design of Coastal Revetments, Seawalls, and Bulkheads. Engineering Manual 1110-2-1614 (1995) – Supersedes portions of Volume 2 of the Shore Protection Manual (SPM).
- Coastal Engineering Manual. Engineer Manual (EM) 1110-2-1100 (2002) – Published in six parts plus an appendix, this set of documents, once complete, will supersede the SPM and EM 1110-2-1614. As of this writing Parts I thru V and the appendix are completed and available. Parts V and VI are considered "Engineering – Based" and present information on design process and selection of appropriate types of solutions to various coastal challenges.

Topic 872 - Planning and Location Studies

872.1 Planning

The development of cost effective protective works requires careful planning. Planning begins with site investigation. The selection of the class of protection can be determined during or following site investigation. For some sites the choice is obvious; at other sites several alternatives or combinations may be applicable. See the FHWA's HDS No. 6, River Engineering for Highway Encroachments for a complete and thorough discussion of hydraulic and environmental design considerations associated with hydraulic structures in moveable boundary waterways.

Some specific site conditions that may dictate selection of a class and type of protection different from those shown in Table 872.1 are:

- Available right of way.
- Available materials.
- Possible damage to other properties through streamflow diversion or increased velocity.
- Environmental concerns.
- Channel capacity or conveyance.
- Conformance to new or existing structures.
- Provisions for side drainage, either surface waters or intersecting streams or rivers.

The first step is to determine the limits of the protection with respect to length, depth and the degree of security required.

Considerations at this stage are:

- The severity of attack.
- The present alignment of the stream or river and potential meander changes.
- The ratio of cost of highway replacement versus cost of protection.
- Whether the protection need be permanent or temporary.
- Analysis of foundation and materials explorations.

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

872.2 Class and Type of Protection

Protective devices are classified according to their function. They are further categorized as to the type of material from which they are constructed or shape of the device. For additional information on specific material types and shapes see Topic 873, Design Concepts.

There are two basic classes of protection, armor treatment and training works. Table 872.1 relates different location environments to these classes of protection.

872.3 Site Consideration

The determination of the lengths, heights, alignment, and positioning of the protection are affected to a large extent by the facility location environment.

An evaluation is required for any proposed highway construction or improvement that encroaches on a floodplain. See Topic 804, Floodplain Encroachments for detailed procedures and guidelines.

(1) Young Valley. Typically young valleys are narrow V-shaped valleys with streams on steep gradients. At flood stage, the stream flow covers all or most of the valley floor. The usual situation for such locations is a structure crossing a well-defined channel in which the design discharge will flow at a moderate to high velocity.

(a) Cross-Channel Location. A cross channel location is a highway crossing a stream on normal or skewed alignment. The erosive forces of parallel flow associated with a normal crossing are generally less of a threat than the impinging and eddy flows associated with a skewed crossing. The effect of constriction by projection of the roadway embankment into the channel should be assessed.

Characteristics to be considered include:

- Stream velocity.
- Scouring action of stream.
- Bank stability.
- Channel constrictions (artificial or natural).
- Nature of flow (tangential or curvilinear).
- Areas of impingement at various stages.
- Security of leading and trailing edges.

Common protection failures occur from:

- Undermining of the toe (inadequate depth/size of foundation), see Figure 872.1 and Table 872.2.
- Local erosion due to eddy currents.
- Inadequate upstream and downstream terminals or transitions to erosion-resistant banks or outcrops.
- Structural inadequacy at points of impingement overtopping.
- Inadequate rock size, see Table 872.2.
- Lack of proper gradation/ layering/ RSP fabric, leading to loss of embankment, see Table 872.2.

Any of the more substantial armor treatments can function properly in such exposures providing precautions are taken to alleviate the probable causes of failure. If the foundation is questionable for concreted-rock or other rigid types it would not be necessary to reject them from consideration but only to provide a more acceptable treatment of the foundation, such as heavy rock or sheet piling.

Figure 872.1

Slope Failure Due to Loss of Toe

Table 872.1

Guide to Selection of Protection

Location	Armor										Training										
	Flexible		Mattresses			Rigid					Guide Dikes Retards & Jetties				Groins				Baffles		
	Vegetation	Riprap	Gabions	Conc. Blocks	Fabric Filled	Grouted Rock	Stacked Conc.	Conc. Lined	Cribs	Bulk heads	Earth	Fencing	Piling	Other	Rock	Grouted Rock	Piling	Other	Drop Structure	Fencing	Rock Earth
Cross Channel																					
Young Valley		X	Ø			X			X	X											
Mature Valley		X	Ø	Ø	Ø	X			X	X	X	X	X	X	X	X			X	X	X
Parallel Encroachment																					
Young Valley		X	Ø			X			X	X											
Mature Valley	X	X	Ø	Ø	Ø	X	Ø		X	X	X	X	X	X	X	X	X	X	X	X	X
Lakes and Tidals Basins	X	X	Ø	Ø	Ø	X	Ø			X											
Ocean Front		X	Ø	Ø						X					X	X	X	X			
Desert-wash																					
Top debris cone		X	Ø	Ø	Ø	X				X											
Center debris cone		X	Ø	Ø	Ø	X													X	X	X
Bottomdebris cone		X	Ø	Ø	Ø	X													X	X	X
Overflow and floodplain	X	X	Ø			X	Ø				X	X	X	X							
Artificial channel	X	X	Ø	Ø	Ø	X	Ø	X													
Culvert																					
Inlet		X				X	Ø			X											
Outlet		X				X	Ø			X											
Bridge																					
Abutment		X		Ø	Ø	X	Ø	X													
Upstream		X				X					X	X	X	X							
Downstream		X				X					X	X	X	X					X		
Roadside ditch	X	X				X	Ø	X													

Ø Where large rock for riprap is not available

Table 872.2

Failure Modes and Effects Analysis for Riprap Revetment

Failure Modes	Effects on Other Components	Effects on Whole System	Detection Methods	Compensating Provisions
Translational slope or slump (slope failure)	Disruption of armor layer	Catastrophic failure	• Mound of rock at bank toe • Unprotected upper bank	• Reduce bank slope • Use more angular or smaller rock • Use granular filter rather than geotextile fabric
Particle erosion (rock undersized)	Loss of armor layer, erosion of filter	Progressive failure	• Rock moved downstream from original location • Exposure of filter	• Increase rock size • Modify rock gradation
Piping or erosion beneath armor (improper filter)	Displacement of armor layer	Progressive failure	• Scalloping of upper bank • Bank cutting • Void beneath and between rocks	• Use appropriate granular or geotextile filter
Loss of toe or key (under designed)	Displacement or disruption or armor layer	Catastrophic failure	• Slumping of rock • Unprotected upper bank	• Increase size, thickness, depth or extent of toe or key

Whether the highway crosses a stream channel on a bridge or over a culvert, economic considerations often lead to constriction of the waterway. The most common constriction is in width, to shorten the structure. Next in frequency is obstruction by piers and bents of bridges or partitions of multiple culverts.

The risk of constricting the width of the waterway is closely related to the relative conveyance of the natural waterway obstructed, the channel scour, and to the channel migration. Constricting the width of flow at structures has the following effects:

- Increase in the upstream water surface elevation (backwater profile).
- Increase in flow velocity through the structure opening (waterway).
- Causes eddy currents around the upstream and downstream ends of the structure.

Unless protection is provided the eddy currents can erode the approach roadway embankment and the accelerated flow can cause scour at bridge abutments. The effects of erosion can be reduced by providing transitions from natural to constricted and back to natural sections, either by relatively short wingwalls or by relatively long training embankments or structures.

Channel changes, if properly designed, can improve conditions of a crossing by reducing skew and curvature and enlarging the main channel. Unfortunately there are "side effects" which actually increase erosion potential. Velocity is almost always increased by the channel change, both by a reduction of channel roughness and increase of slope due to channel shortening. In addition, channel changes affecting stream gradient may have upstream and/or downstream effects as the stream adjusts in relation to its sediment load.

At crossing locations, lateral erosion can be controlled by positive protection, such as armor on the banks, rock spurs to deflect currents away from the banks, retards to reduce riparian velocity, or vertical walls or bulkheads. The life cycle cost of such devices should be considered in the economic studies to choose a bridge length which minimizes total cost.

Accurate estimates of anticipated scour depths are a prerequisite for safe, cost effective designs. Design criteria require that bridge foundations be placed below anticipated scour depths. For this reason the design of protection to control scour at such locations is seldom necessary for new construction. However, if scour may undercut the toes of dikes or embankments positive methods including self-adjusting armor at the toe, jetties or retards to divert scouring currents away from the toe, or sill-shaped baffles interrupting transport of bedloads should be considered.

There is the potential for instability from saturated or inundated embankments at crossings with embankments projecting into the channel. Failures are usually reported as "washouts", but several distinct processes should be noted:

- Saturation of an embankment reduces its angle of repose. Granular fills with high permeability may "dissolve" steadily or slough progressively. Cohesive fills are less permeable, but failures have occurred during falling stages.
- As eddies carve scallops in the embankment, saturation can be accelerated and complete failure may be rapid. Partial or total losses can occur due to an upstream eddy, a downstream eddy, or both eddies eroding toward a central conjunction. Training devices or armor can be employed to prevent damage.
- If the fill is pervious and the pavement overtopped, the buoyant pressure under the slab will exceed the weight of slab and shallow overflow by the pressure head of the hydraulic drop at the

shoulder line. A flat slab of thickness, t, will float when the upstream stage is 4t higher than the top of the slab. Thereafter the saturated fill usually fails rapidly by a combination of erosion and sloughing. This problem can occur or be increased when curbs, dikes, or emergency sandbags maintain a differential stage at the embankment shoulder. It is increased by an impervious or less pervious mass within the fill. Control of flotation, insofar as bank protection is concerned, should be obtained by using impervious armor on the upstream face of the embankment and a pervious armor on the downstream face.

Culvert problem locations generally occur in and along the downstream transition. Sharp divergence of the high velocity flow develops outward components of velocity which attack the banks directly by impingement and indirectly by eddies entrained in quieter water. Downward components and the high velocity near the bed cause the scour at the end of the apron.

Standard plans of warped wingwalls have been developed for a smooth transition from the culvert to a trapezoidal channel section. A rough revetment extension to the concrete wingwalls is often necessary to reduce high velocity to approximate natural flow. Energy dissipaters may be used to shorten the deceleration process when such a transition would be too long to be economical. Bank protection at the end of wingwalls is more cost effective in most cases.

(b) Parallel Location. With parallel locations the risk of erosion damage along young streams increases where valleys narrow and gradients steepen. The risk of erosion damage is greatest along the outer bend of natural meanders or where highway embankment encroaches on the main channel.

The *encroaching* parallel location is very common, especially for highways following mountain streams in narrow young valleys or canyons. Much of the roadway is supported on top of the bank or a berm and the outer embankment encroaches on the channel in a zone of low to moderate velocity. Channel banks are generally stable and protection, except at points of impingement, is seldom necessary.

The *constricting* parallel location is an extreme case of encroaching location, causing such impairment of channel that acceleration of the stream through the constriction increases its attack on the highway embankment requiring extra protection, or additional waterway must be provided by deepening or widening along the far bank of the stream.

In young valleys, streams are capable of high velocity flows during flood stages that may be damaging to adjacent highway facilities. Locating the highway to higher ground or solid support is always the preferred alternative when practical.

Characteristics to be considered include:

- High velocity flow.
- Narrow confined channels.
- Accentuated impingement.
- Swift overflow.
- Disturbed flow due to rock outcrops on the banks or within the main channel.
- Alterations in flow patterns due to the entrance of side streams into the main channel.

Protective methods that have proven effective are:

- Rock slope protection.
- Concreted-rock slope protection.
- Walls of masonry and concrete.
- Articulated concrete block revetments.
- Sacked concrete.
- Cribs walls of various materials.

(2) Mature Valley. Typically mature valleys are broad V-shaped valleys with associated

floodplains. The gradient and velocity of the stream are low to moderate. In addition to the general information previously given, the following applies to mature valleys.

(a) Cross-Channel Location. The usual situation is a structure crossing a braided or meandering normal flow channel. The marginal area subject to overflow is usually traversed by the highway on a raised embankment and may have long approaches extending from both banks.

Characteristics to be considered include:

- Shifting of the main channel.
- Skew of the stream to the structure.
- Foundation in deep alluvium.
- Erodible embankment materials.
- Channel constrictions, either artificial or natural, which may affect or control the future course of the stream.
- Variable flow characteristics at various stages.
- Stream acceleration at the structure.

Armor protection has proven effective to prevent erosion of road approach embankments, supplemented if necessary by stream training devices such as guide dikes, permeable retards or jetties to direct the stream through the structure. The abutments should not depend on the training dikes to protect them from erosion and scour. At bridge ends one of the more substantial armor types may be required, but bridge approach embankments affected only by overflow seldom require more than a light revetment, such as a thin layer of rocky material, vegetation, or a fencing along the toe of slope. For channel flow control upstream, the size and type of training system ranges from pile wings for high velocity, through permeable jetties for moderate velocity, to the earth dike suitable for low velocity.

The more common failures in this situation occur from:

- Lack of upstream control of channel alignment.
- Damage of unprotected embankments by overflow and return flow.
- Undercut foundations.
- Formation of eddies at abrupt changes in channel.
- Stranding of drift in the converging channel.

(b) Parallel Location. Parallel highways along mature rivers are often situated on or behind levees built, protected and maintained by other agencies. Along other streams, rather extensive protective measures may be required to control the action of these meandering streams.

Channel change is an important factor in locations parallel to mature streams. The channel change may be to close an embayment, to cut off an oxbow, or to shift the alignment of a long reach of a stream. In any case, positive means must be adopted to prevent the return of the stream to its natural course. For a straight channel, the upstream end is critical, usually requiring bank protection equivalent to the facing of a dam. On a curved channel change, all of the outer bend may be critical, requiring continuous protection. For a channel much shorter than the natural channel, particularly for elimination of an oxbow, the corresponding increase in gradient may require transverse weirs as grade control structures to prevent undercutting. For unusual channel changes, preliminary plans and hydraulic data must be submitted to FHWA for approval (see Index 805.5).

(3) Lakes and Tidal Basins. Highways adjacent to lakes or basins may be at risk from wave generated erosion. All bodies of waters generate waves. Height of waves is a function of fetch and depth. Erosion along embankments behind shallow coves is reduced because the higher waves break upon reaching a shoal in shallow water. The threat of erosion in deep water at headlands or along causeways is increased. Constant exposure to even the

rippling of tiny waves may cause severe erosion of some soils.

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

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

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

Beach protection considerations include:

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

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

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

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

Seasonal shifts of the shore line result from combinations of:

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

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

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

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

See Index 873.3(2) for further discussion on determining the size of rocks necessary in shore protection for various wave heights.

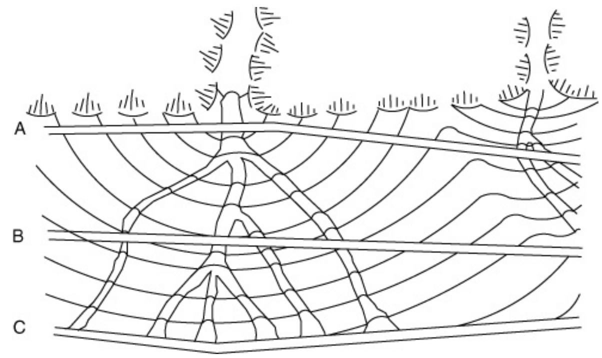
(5) *Desert Wash Locations.* Special consideration should be given to highway locations across the natural geographical features of desert washes, sand dunes, and other similar regions.

Desert washes are a prominent feature of the physiography of California. Many long stretches of highway are located across a succession of outwash cones. Infrequent discharge is typically wide and shallow, transporting large volumes of solids, both mineral and organic. Rather than bridge the natural channels, the generally accepted technique is to concentrate the flow by a series of guide dikes leading like a funnel to a relatively short crossing.

An important consideration at these locations is instability of the channel, see Figure 872.2. For a location at the top of a cone (Line A), discharge is maximum, but the single channel emerging from the uplands is usually stable. For a location at the bottom of the cone (Line C), instability is maximum with poor definition of the channel, but discharge is reduced by infiltration and stream dispersion. The energy of the stream is usually dissipated so that any protection required is minimal. The least desirable location is midway between top and bottom (Line B), where large discharge may approach the highway in any of several old channels or break out on a new line. Control may require dikes continuously from the top of the cone to such a mid-cone site with slope protection added near the highway where the converging flow is accelerated. See Figure 872.3, which depicts a typical alluvial fan.

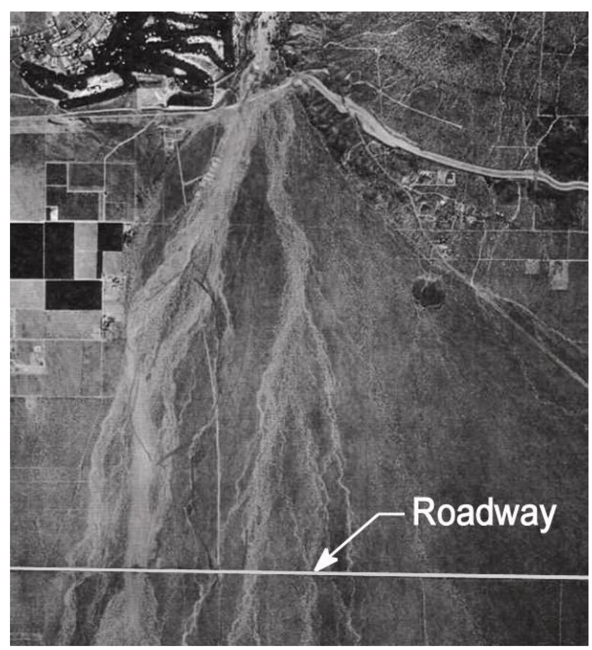
Also common are roadway alignments which longitudinally encroach, or are fully within the desert wash floodplain, see Figure 872.4. Re-alignment to a stable location should be the first consideration, but restrictions imposed by federal or state agencies (National Park Service, USDA Forest Service, etc.) may preclude that option, somewhat similar to transverse crossings. The designer may need to consider allowing frequent overtopping and increased sediment removal maintenance since an "all weather design" within these regimes can often lead to large scale roadway washout.

Figure 872.2
Alternative Highway Locations Across Debris Cone



A. Cross at a single definite channel
B. A series of unstable indefinite channels and
C. A widely dispersed and diminished flow

Figure 872.3
Alluvial Fan



Typical multi-channel stream threads on alluvial fan. Note location of roadway crossing unstable channels.

Figure 872.4
Desert Wash Longitudinal Encroachment

Road washout due to longitudinal location in desert wash channel

Characteristics to be considered include:

- The intensity of rainfall and subsequent run-off.
- The relatively large volumes of solids that are carried in such run-off.
- The lack of definition and permanence of the channel.
- The scour depths that can be anticipated.
- The lack of good foundation.

Effective protective methods include armor along the highway and at structures and the probable need for baffles to control the direction and velocity of flow. Installations of rock, fence, palisades, slope paving, and dikes have been successful.

The Federal Emergency Management Agency (FEMA) Flood Hazard Mapping website contains information on recognizing alluvial fan landforms and methods for defining active and inactive areas. See their "Guidelines for Determining Flood Hazards on Alluvial Fans" at http://www.fema.gov/fhm/ft_tocs.shtm.

872.4 Data Needs

The types and amount of data needed for planning and analysis of bank and shore protection varies from project to project depending upon the class and extent of the proposed protection, site location environment, and geographic area. The data that is collected and developed including preliminary calculations, and alternatives considered should be documented in project development reports (Environmental Document, Project Report, etc.) or as a minimum in the project file. These records serve to guide the detailed designs, and provide reference background for analysis of environmental impacts and other needs such as permit applications and historical documentation for any litigation which may arise.

Recommendations for data needs can be requested from the District Hydraulics Engineer or determined from Chapter 8 of FHWA's HDS No. 6, for a more complete discussion of data needs for highway crossings and encroachments on rivers. Further references to data needs are contained in Chapter 810, Hydrology and FHWA's HDS No. 2, Highway Hydrology.

Topic 873 - Design Concepts

873.1 Introduction

No attempt will be made here to describe in detail all of the various devices that have been used to protect embankments against scour. Methods and devices not described may be used when justified by economical analysis. Not all publicized treatments are necessarily suited to existing conditions for a specific project.

A set of plans and specifications must be prepared to define and describe the protection that the design engineer has in mind. These plans should show controlling factors and an end product in such detail that there will be no dispute between the construction engineer and contractor. To serve the dual objectives of adequacy and economy, plans and specifications should be precise in defining materials to be incorporated in the work, and flexible in describing methods of construction or conformance of the end product to working lines and grades.

Recommendations on channel lining, slope protection, and erosion control materials can be requested from the District Hydraulic Engineer, the District Materials Branch and the Office of State Highway Drainage Design in Headquarters. The District Landscape Architect will provide recommendations for temporary and permanent erosion and sediment control measures. The

Caltrans Bank and Shore Protection Committee is available on request to provide expert advice on extraordinary situations or problems and to provide evaluation and formal approvals for acceptable non-standard designs. See Index 802.3 for further information on the organization and functions of the Committee.

Combinations of armor-type protection can be used, the slope revetment being of one type and the foundation treatment of another. The use of rigid, non-flexible slope revetment may require a flexible, self-adjusting foundation for example: concreted-rock on the slope with heavy rock foundation below, or PCC slope paving with a steel sheet-pile cutoff wall for foundation.

Bank protection may be damaged while serving its primary purpose. Lower cost replaceable facilities may be more economical than expensive permanent structures. However, an expensive structure may be economically warranted for highways carrying large volumes of traffic or for which no detour is available.

Cost of stone is extremely sensitive to location. Variables are length of haul, efficiency of the quarry in producing acceptable sizes, royalty to quarry and, necessity for stockpiling and rehandling. On some projects the stone may be available in roadway excavation.

873.2 Design High Water and Hydraulics

The most important, and often the most perplexing obligation, in the design of bank and shore protection features is the determination of the appropriate design high water elevation to be used. The design flood stage elevation should be chosen that best satisfies site conditions and level of risk associated with the encroachment. The basis for determining the design frequency, velocity, backwater, and other limiting factors should include an evaluation of the consequences of failure on the highway facility and adjacent property. Stream stability and sediment transport of a watercourse are critical factors in the evaluation process that should be carefully weighted and documented. Designs should not be based on an arbitrary storm or flood frequency.

A suggested starting point of reference for the determination of the design high water level is that the protection withstands high water levels caused by meteorological conditions having a recurrence interval of one-half the service life of the protected facility. For example, a modern highway embankment can reasonably be expected to have a service life of 100 years or more. It would therefore be appropriate to base the preliminary evaluation on a high water elevation resulting from a storm or flood with a 2 percent probability of exceedance (50 year frequency of recurrence). The first evaluation may have to be adjusted, either up or down, to conform with a subsequent analysis which considers the importance of the encroachment and level of related risks.

There is always some risk associated with the design of protection features. Special attention must be given to life threatening risks such as those associated with floodplain encroachments. Significant floodplain risks are classified as those having probability of:

- Catastrophic failure with loss of life.
- Disruption of fire and ambulance services or closing of the only evacuation route available to a community.

Refer to Topic 804, Floodplain Encroachments, for further discussion on evaluation of risks and impacts.

(1) *Streambank Locations.* The velocity along the banks of watercourses with smooth or uniformly rough tangent reaches may only be a small percentage of the average stream velocity. However, local irregularities of the bank and streambed may cause turbulence that can result in the bank velocity being greater than that of the central thread of the stream. The location of these irregularities is not always permanent as they may be caused by local scour, deposition of rock and sand, or stranding of drift during high water changes. It is rarely economical to protect against all possibilities and therefore some damage should always be anticipated during high water stages.

    Essential to the design of streambank protection is sufficient information on the characteristics of the watercourse under consideration. For proper analysis, information on the following types of watercourse characteristics must be developed or obtained:

- Design Discharge
- Design High Water Level
- Flow Types
- Channel Geometry
- Flow Resistance
- Sediment Transport

Refer to Chapter 810, Hydrology, for a general discussion on hydrologic analysis and specifically to Topic 817, Flood Magnitudes; Topic 818, Flood Probability and Frequency; and Topic 819, Estimating Design Discharge. For a detailed discussion on the fundamentals of alluvial channel flow, refer to Chapter 3, HDS No. 6, and to Chapter 4, HDS No. 6, for further information on sediment transport.

(2) Ocean & Lake Shore Locations. Information needed to design shore protection is:

- Design High Water Level
- Design Wave Height

(a) Design High Water Level. The flood stage elevation on a lake or reservoir is usually the result of inflow from upland runoff. If the water stored in a reservoir is used for power generation, flood control, or irrigation, the design high water elevation should be based on the owners schedule of operation.

Except for inland tidal basins affected by wind tides, floods and seiches, the static or still-water level used for design of shore protection is the highest tide. In tide tables, this is the stage of the highest tide above "tide-table datum" at MLLW. To convert this to MSL datum there must be subtracted a datum equation (2.5 feet to 3.9 feet) factor. If datum differs from MSL datum, a further correction is necessary. These steps should be undertaken with care and independently checked. Common errors are:

- Ignoring the datum equation.
- Adding the factor instead of subtracting it.
- Using half the diurnal range as the stage of high water.

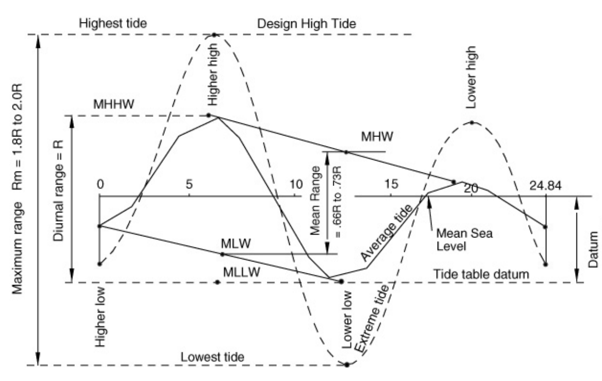
To clarify the determination of design high-water, Fig. 873.2A shows the *Highest Tide* in its relation to an extreme-tide cycle and to a hypothetical average-tide cycle, together with nomenclature pertinent to three definitions of tidal range. Note that the cycles have two highs and two lows. The average of all the higher highs for a long period (preferably in multiples of the 19-yr. metonic cycle) is MHHW, and of all the *lower* lows, MLLW. The vertical difference between them is the *diurnal range*.

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

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

See Index 814.5, Tides and Waves, for information on where tide and wave data may be obtained.

Figure 873.2A
Nomenclature of Tidal Ranges



Because of the great variation of tidal elements, Figure 873.2A was not drawn to scale.

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

(b) Design Wave Heights.

(1) General. Even for the simplest of cases, the estimation of water levels caused by meteorological conditions is complex. Elaborate numerical models requiring the use of a computer are available, but simplified techniques may be used to predict acceptable wind wave heights for the design of highway protection facilities along the shores of embayments, inland lakes, and reservoirs. It is recommended that for ocean shore protection designs the assistance of the U.S. Army Corp of Engineers be requested.

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

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

The prediction of wave heights from boat generated waves must be estimated from observations.

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

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

$$H_{10} = 1.27\ H_s \quad \text{and} \quad H_1 = 1.67\ H_s$$

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

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

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

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

(a) Hindcasting -- The U.S. Army Corp of Engineers has historical records of onshore and offshore weather and wave observations for most of the California coastline. Design wave height predictions for coastal shore protection facilities should be made using this information and hindcasting methods. Deep-water ocean wave characteristics derived from offshore data analysis may need to be transformed to the project site by refraction and diffraction techniques. As mentioned previously, it is strongly advised that the Corps technical expertise be obtained so that the data are properly interpreted and used.

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

The following assumptions pertain to these simplified methods:

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

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

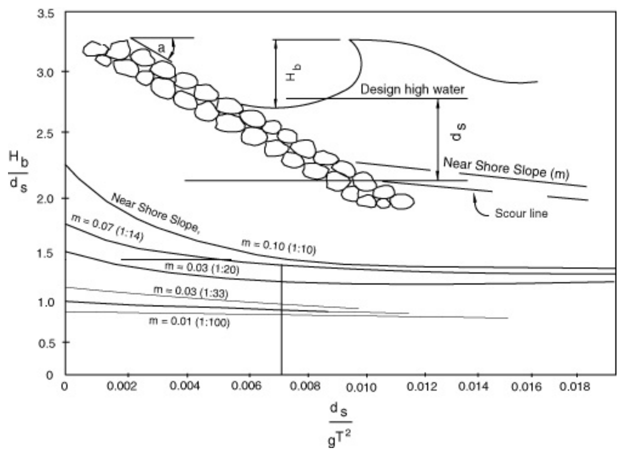
The applicability of a wave forecasting method depends on the available wind data, water depth, and overland topography. Water depth affects wave generation and for a given set of wind and fetch conditions, wave heights will be smaller and wave periods shorter if the wave generation takes place in transitional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

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

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

An initial estimate of wind generated significant wave heights can be made by using Figure 873.2B. If the estimated wave height from the nomogram is greater than 2 feet, the procedure may need to be refined. It is recommended that advice from the Army Corps of Engineers be obtained to refine significant wave heights, H_S, greater than 2 feet.

Figure 873.2C
Design Breaker Wave



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

<u>Solution</u>

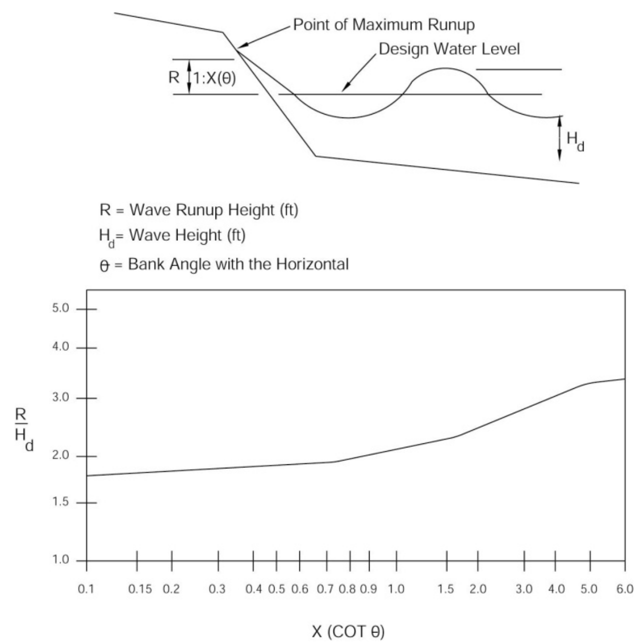
$$\frac{d_s}{gT^2} = \frac{2\text{ ft}}{(32.2\text{ ft/s}^2) \times (3\text{ sec})^2} = 0.007$$

From Graph) - $H_b/d_s = 1.4$
$H_b = 2 \times 1.4 = 2.8$ ft
<u>Answer</u>
Since the maximum breaker wave height, H_b, is smaller than the significant deepwater wave height, H_s, the design wave height H_d is 2.8 feet.
T = Wave Period (SPM)

(c) Littoral Processes. Littoral processes result from the interaction of winds, waves, currents, tides, and the availability of sediment. The rates at which sediment is supplied to and removed from the shore may cause excessive accretion or erosion that can affect the structural integrity of shore protection structures or functional usefulness of a beach. The aim of good shore protection design is to maintain a stable shoreline where the volume of sediment supplied to the shore balances that which is removed.

Designers interested in a more complete discussion on littoral processes should consult the U.S. Army Corps of Engineers' Coastal Engineering Manual (CEM) – Part III.

Figure 873.2D
Wave Run-up on Smooth Impermeable Slope



873.3 Armor Protection

(1) *General.* Armor is the artificial surfacing of bed, banks, shore or embankment to resist erosion or scour. Armor devices can be flexible (self adjusting) or rigid.

Hard armoring of stream banks and shorelines, primarily with rock slope protection (RSP), has been the most common means of providing long-term protection for transportation facilities, and most importantly, the traveling public. With many years of use, dozens of formal studies and thousands of constructed sites, RSP is the armor type for which there exists the most quantifiable data on performance, constructability, maintainability and durability, and for which there exist several nationally recognized design methods.

Due to the above factors, RSP is the general standard against which other forms of armoring are compared. The results of internal research led to the publication of Report No. FHWA-CA-TL-95-10, “California Bank and Shore Rock Slope Protection Design”. Within that report, the methodology for RSP design adopted as the Departmental standard, is the California Bank and Shore, (CABS), layered design. The full report is available at the following website:

http://www.dot.ca.gov/hq/oppd/hydrology/hydroidx.htm.

This design method, which is applied with slight variation to ocean and lake shores vs. stream banks, and is also followed for concreted RSP designs, is the only protection method as of this writing that has been formally adopted by the Caltrans Bank and Shore Protection Committee. Section 72 of the Standard Specifications provides all construction and material specifications for RSP designs. While standards (i.e., Standard Plans, Standard Specifications and/or SSP’s) do exist for some other products discussed in this Chapter (most notably for gabions, but also for certain rolled or mat-style erosion control products), their primary application is for relatively flat slope or shallow ditch erosion control (gabions are also used as an earth retaining structure, see Topic 210 for more details).

Other armor types listed below and described throughout this Chapter are viable and may be used, upon approval of the Headquarters Hydraulic Engineer or Caltrans Bank and Shore Protection Committee, where conditions warrant. Although the additional step of headquarters approval of these non-standard designs is required, designers are encouraged to consider alternative designs, particularly those that incorporate vegetation or products naturally present in stream environments. The District Landscape Architect can provide design assistance together with specifications and details for the vegetative portion of this work.

(a) Flexible Types.

- Rock slope protection.
- Broken concrete slope protection.
- Broken concrete, uncoursed.
- Gabions, Standard Plan D100A and D100B.
- Precast concrete articulated blocks.
- Rock filled cellular mats.

(b) Rigid Types.

- Concreted-rock slope protection.
- Sacked concrete slope protection.
- Concrete slope protection.
- Concrete filled fabric slope protection.
- Air-blown mortar.
- Soil cement slope protection.

(c) Other Armor types:

(1) Channel Liners and Vegetation. Temporary channel lining can be used to promote vegetative growth in a drainage way or as protection prior to the placement of permanent armoring. This type of lining is used where an ordinary seeding and mulch application would not be expected to withstand the force of the channel flow. In addition to the following, other suitable products of natural or synthetic materials are available that may be used as temporary or permanent channel liners.

- Excelsior
- Jute
- Paper mats
- Fiberglass roving
- Geosynthetic mats or cells
- Pre-cast concrete blocks with open cells
- Brush layering
- Rock riprap in sizes smaller than backing No. 3

(2) Bulkheads. The bulkhead types are steep or vertical structures, like retaining walls, that support natural slopes or constructed embankments which include the following:

- Gravity or pile supported concrete or masonry walls.
- Crib walls

- Sheet piling
- Sea Walls

(d) General Design Criteria. In selecting the type of flexible or rigid armor protection to use the following characteristics are important design considerations.

(1) The lower limit, or toe, of armor should be below anticipated scour or on bedrock. If for any reason this is not economically feasible, a reasonable degree of security can be obtained by placement of additional quantities of heavy rock at the toe which can settle vertically as scour occurs.

(2) In the case of slope paving or any expensive revetment which might be seriously damaged by overtopping and subsequent erosion of underlying embankment, extension above design high water may be warranted. The usual limit of extension for streambank protection above design high water is 1 foot to 2 feet in unconstricted reaches and 2 feet to 3 feet in constricted reaches.

(3) The upstream terminal can be determined best by observation of existing conditions and/or by measuring velocities along the bank.

The terminal should be located to conform to outcroppings of erosion-resistant materials, trees, shrubs or other indications of stability.

In general, the upstream terminal on bends in the stream will be some distance upstream from the point of impingement or the beginning of curve where the effect of erosion is no longer damaging.

(4) When possible the downstream terminal should be made downstream from the end of the curve and against outcroppings, erosion-resistant materials, or returned securely into the bank so as to prevent erosion by eddy currents and velocity changes occurring in the transition length.

(5) The encroachment of embankment into the stream channel must be considered with respect to its effect on the conveyance of the stream and possible damaging effect on properties upstream due to backwater and downstream due to increased stream velocity or redirected stream flow.

(6) A smooth surface will generally accelerate velocity along the bank, requiring additional treatment (e.g., extended transition, cut-off wall, etc.) at the downstream terminal. Rougher surfaces tend to keep the thread of the stream toward the center of the channel.

(7) Heavy-duty armor used in exposures along the ocean shore may be influenced or dictated by economics, or the feasibility of handling heavy individual units.

(2) Flexible Revetments.

(a) Streambank Rock Slope Protection.

(1) General Features. This kind of protection, commonly called riprap, consists of rock courses placed upon the embankment or the natural slope along a stream. Rock, as a slope protection material, has a number of desirable features which have led to its widespread application.

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

- It is flexible and is not impaired nor weakened by slight movement of the embankment resulting from settlement or other minor adjustments.
- Local damage or loss is easily repaired by the addition of similar sized rock where required.
- Construction is not complicated and special equipment or construction practices are not usually necessary. (Note that Method A placement of

very large rock may require large cranes or equipment with special lifting capabilities).

- Appearance is natural, and usually acceptable in recreational and scenic areas.
- If exposed to fresh water, vegetation may be induced to grow through the rocks adding structural value to the embankment material and restoring natural roughness.
- Additional thickness (i.e., mounded toe design) can be provided at the toe to offset possible scour when it is not feasible to found it upon bedrock or below anticipated scour.
- Wave run-up is less than with smooth types (See Figure 873.2D).
- It is salvageable, may be stockpiled and reused if necessary.

In designing the rock slope protection for a given embankment the following determinations are to be made for the typical section.

- Depth at which the stones are founded (bottom of toe trench).
- Elevation at the top of protection.
- Thickness of protection.
- Need for geotextile and backing material.
- Face slope.

(a) Placement -- Two different methods of placement for rock slope protection are allowed under Section 72 of the Standard Specifications: Placement under Method A requires considerable care, judgment, and precision and is consequently more expensive than Method B. Method A should be specified primarily where large rock is required, but also for relatively steeper slopes.

Under some circumstances the costs of placing rock slope protection with refinement are not justified and Method B placement can be specified. To compensate for a partial loss and assure stability and a reasonably secure protection, the thickness is increased over the more precise Method A by 25 percent.

(b) Foundation Treatment -- The foundation excavation must afford a stable base on bedrock or extend below anticipated scour.

Terminals of revetments are often destroyed by eddy currents and other turbulence because of nonconformance with natural banks. Terminals should be secured by transitions to stable bank formations, or the end of the revetment should be reinforced by returns of thickened edges.

While a significant amount of research is currently being conducted, few methods exist for estimating scour along stream banks. One of the few is the method contained in the CHANLPRO Program developed by the U.S. Army Corps of Engineers. Based on the flume studies at the Corps' Waterways Experiment Station, the program is primarily used by the Corps for RSP designs on streams with 2 percent or lesser gradients, but contains an option for scour depth estimates in bends for sand channels. CHANLPRO is available at the following USACE website: http://chl.erdc.usace.army.mil/CHL.aspx?p=s&a=Software;3 along with a user guide containing equations, charts, assumptions and limitations to the method and example problems.

(c) Embankment Considerations -- Embankment material is not normally carried out over the rock slope protection so that the rock becomes part of the fill. With this type of construction fill material can filter down through the voids of the large stones and that portion of the fill above the rocks could be lost. If it is necessary to carry embankment material out over the rock slope protection a geotextile is required to prevent the loses of fill material.

The embankment fill slope is usually determined from other considerations such as the angle of repose for embankment material, or the normal 1V:4H specified for high-standard roads. If the necessary size of rock for the given exposure is not locally available, consideration should be given to flattening of the embankment slope to allow a smaller size stone, or substitution of other types of protection. On high embankments, alternate sections on several slopes should be compared, practically and economically; flatter slopes require smaller stones in thinner sections, but at the expense of longer slopes, a lower toe elevation, increased embankment, and perhaps additional right of way.

Where the roadway alignment is fixed, slope flattening will often increase embankment encroachment into the stream. When such an encroachment is environmentally or technically undesirable, the designer should consider various vertical, or near vertical, wall type alternatives to provide adequate stream width, allowing natural channel migration and the opportunity for enhancing habitat.

(d) Rock Slope Protection Fabric and Inner Layers of Rock -- The layered method of designing RSP installations was developed prior to widespread availability of the rock slope protection fabrics which are described in Standard Specification Section 88. The RSP fabric and multiple layers of rock ensure that fine soil particles do not migrate through the RSP due to hydrostatic forces and, thus, eliminate the potential for bank failure. The use of RSP fabric provides an inexpensive layer of protection retaining embankment fines in lieu of placing backing No. 3 or similar small, well graded materials. See Index 873.3(2)(a)(1)(e) "Gravel Filter."

Under special circumstances, the designer may consider allowing holes to be cut in the RSP fabric, generally to facilitate more rapid/extensive rooting of woody vegetation through the RSP revetment. This practice is only necessary for deeply rooted plant species. Holes in RSP fabric should not be cut below the stage of the 2-year return period event. The District Hydraulic Unit should be consulted for advice prior to any determination to cut or otherwise modify standard installation of RSP fabric.

Additionally, stronger and heavier RSP fabrics than those listed in the Standard Specifications are manufactured. They are used in special designs for larger than standard RSP sizes, or emergency installations where placement of the layered design is not feasible and large RSP must be placed directly on the fabric. These heavy weight fabrics have unit weights of up to 16 ounces per square yard. Contact the Headquarters Hydraulic Engineer for assistance regarding usage applications of heavy weight RSP fabrics.

(e) Gravel Filter -- Generally RSP fabric should always be used unless there is a permit requirement for establishment of vegetation that precludes the placement of fabric due to inadequate root penetration. Where RSP fabric cannot be placed, such as in stream environments where CA Fish & Game and NOAA Fisheries strongly discourage the use of RSP Fabric, a gravel filter is usually necessary with most native soil conditions to stop fines from bleeding through the typical RSP classes.

When a gravel filter is to be placed, the designer is advised to work with the District Materials Office to get a recommendation for the necessary gradation to work effectively with both the native backfill and the base layer of the RSP that is being placed. Among the methods available for designing the gravel filter are the Terzaghi method, developed exclusively for situations where the native backfill is sand, and the Cisten-Ziems method, which is often used for a broad variety of soil types. Where streambanks must be significantly rebuilt and reconfigured with imported material before RSP placement, the designer must ensure that the imported material will not bleed through the designed gravel filter.

(2) Streambank Protection Design. In the lower reaches of larger rivers wave action resulting from navigation or wind blowing over long reaches may be much more serious than velocity. A 2 foot wave, for example, is more damaging than direct impingement of a current flowing at 10 feet per second.

Well designed streambank rock slope protection should:

- Assure stability and compatibility of the protected bank as an integral part of the channel as a whole.
- Connect to natural bank, bridge abutments or adjoining improvements with transitions designed to ease differentials in alignment, grade, slope and roughness of banks.
- Eliminate or ease local embayments and capes so as to streamline the protected bank.
- Consider the effects of backwater above constrictions, superelevations on bends, as well as tolerance of occasional overtopping.
- Not be placed on a slope steeper than 1.5H:1V. Flatter slopes (see Figure 873.3A) use lighter stones in a thinner section and encourage overgrowth of vegetation, but may not be permissible in narrow channels.
- Use stone of adequate weight to resist erosion, derived from Figure 873.3A.
- Prevent loss of bank materials through interstitial spaces of the revetment. Rock slope protection fabric and multiple layers of backing should be used.
- Rest on a good foundation on bedrock or extend below the depth of probable scour. If questionable, use heavy bed stones and provide a wide base section with a reserve of material to slough into local scour holes (i.e., mounded toe).
- Reinforce critical zones on outer bends subject to impinging flow, using heavier stones, thicker section, and deeper toe.
- Be constructed in two or more layers of rock sizes, with progressively smaller rock toward native bank to prohibit loss of soil fines.
- Be constructed of rock of such shape as to form a stable protection

structure of the required section. Rounded boulders or cobbles must not be used on prepared ground surfaces having slopes steeper than 2.5H:1V

(a) Stone Size -- Where stream velocity governs, rock size may be estimated by using the nomogragh, Figure 873.3A.

The nomograph is derived from the following formula:

$$W = \frac{0.00002V^6 sg_r \csc^3(\beta - \alpha)}{(sg_r - 1)^3}$$

Where:

sg_r = specific gravity of stones

α = angle of face slope from the horizontal

β = 70 for broken rock, a constant

W = weight of minimum stable stone in lbs

V = 2/3 average stream velocity, fps (flow parallel to bank) or 4/3 average stream velocity, fps (flow impinging on bank)

Where wave action is dominant, design of rock slope protection should proceed as described for shore protection.

(b) Design Height -- The top of rock slope protection along a stream bank should be carried to the elevation of the design high water plus some allowance for freeboard. The flood stage elevation adopted for design may be based on an empirically derived frequency of recurrence (probability of exceedance) or historic high water marks. This stage may be exceeded during infrequent floods, usually with little or no damage to the upper slope.

Design high water should not be based on an arbitrary storm frequency alone, but should consider the cost of carrying the protection to this height, the probable duration and damage if overtopped, and the importance of the facility.

When determining freeboard, or the height above design high water from which the RSP is to extend, one should consider: the size and nature of debris in the flow; the resulting potential for damage to the bank, the potential for streambed aggradation; and the confidence in data used to estimate design highwater. Freeboard may also be affected by regulatory or local agency requirements. Freeboard may be more generous along freeways, on bottleneck routes, on the outside bends of channels, or around critical bridges.

Design high water should be adjusted to the site based on sound engineering judgment.

Design Example -- The following example reflects the CABS method for designing RSP as described in Report No. FHWA – CA – TL – 95 – 10, as well as identify some of the considerations and technical principles that the designer must address to complete the installation design. These same considerations and principles apply to concreted RSP as well as RSP placed on beaches and shores (which are covered later), and therefore, separate examples for those designs are not provided. The designer is encouraged to review the entire report referenced above, available on the Division of Design website, for a comprehensive discussion of the basis of the CABS method and

Figure 873.3A

Nomograph of Stream-Bank Rock Slope Protection

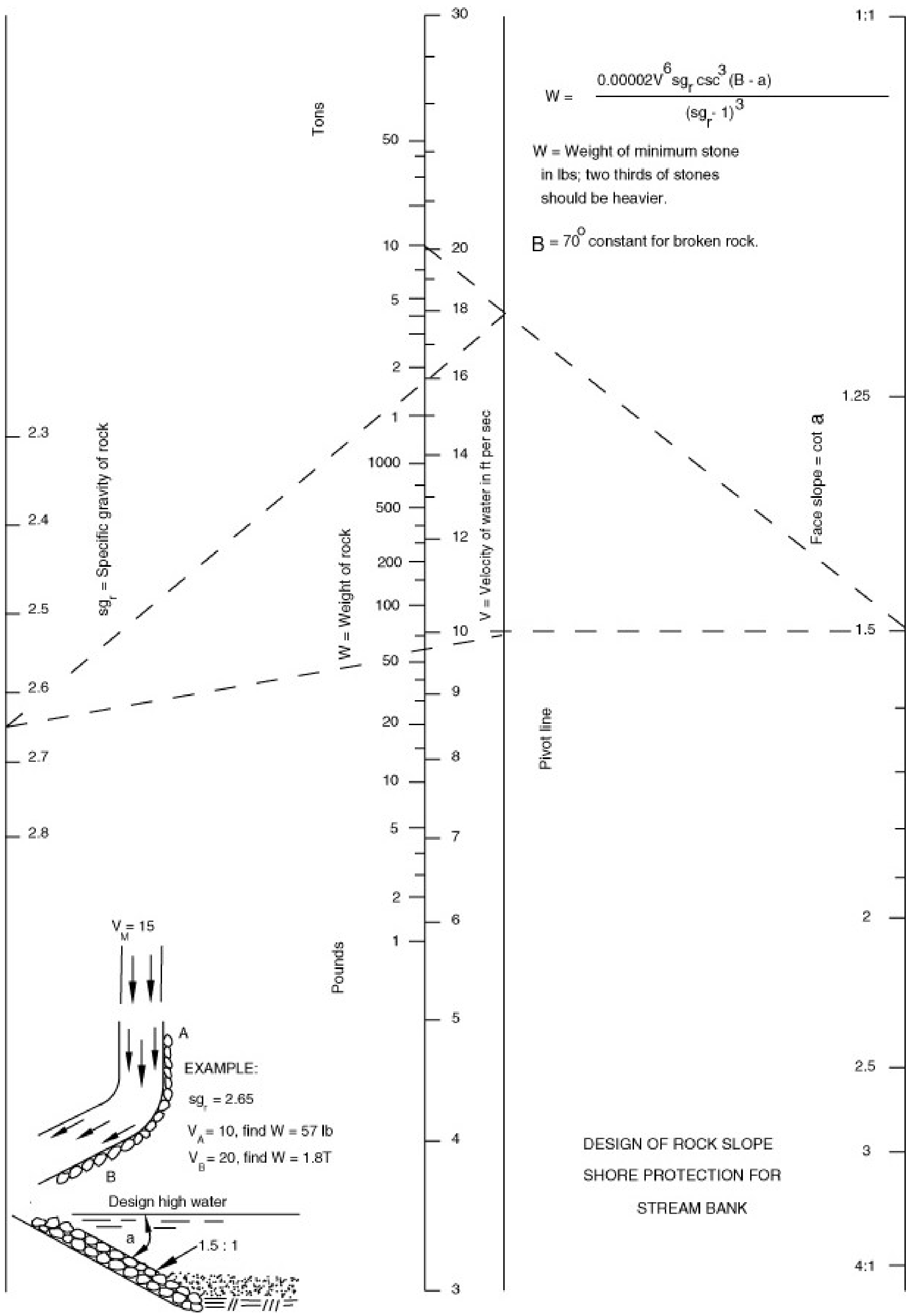
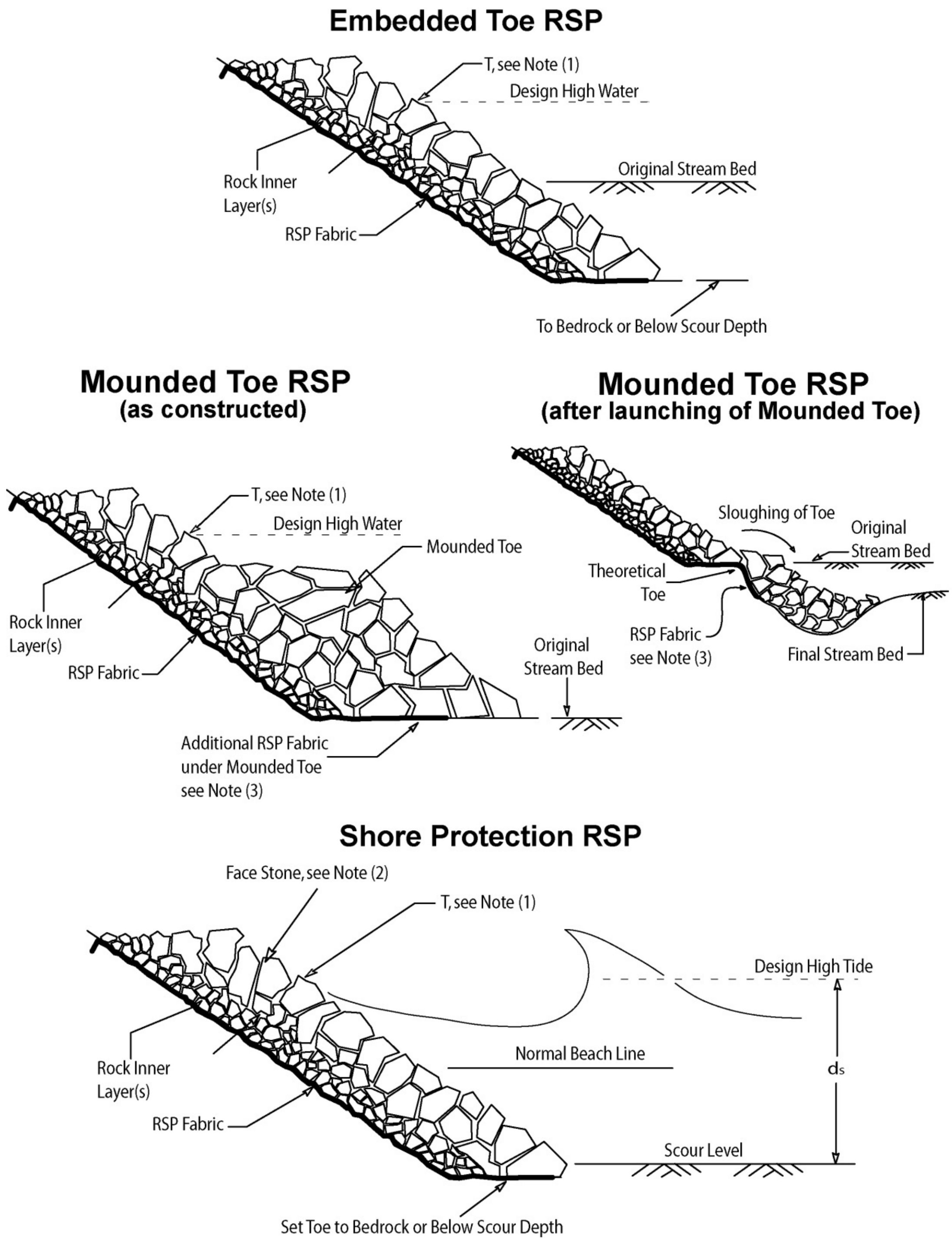


Figure 873.3C
Rock Slope Protection



NOTES:

(1) Thickness "T" from Table 873.3 C.
(2) Face stone is determined from Figure 873.3G.
(3) RSP fabric not to extend more than 20 percent of the base width of the Mounded Toe past the Theoretical Toe.

RSP design considerations. The following example assumes that the designer has conducted the appropriate site assessments and resulting calculations to establish average stream velocity, estimated depth of scour, stream alignment (i.e., parallel or impinging flow), length of stream bank to be protected and locations of natural hard points (e.g., rock outcroppings). Field reviews and discussions with maintenance staff familiar with the site are critical to the success of the design.

Given for example:

- Average stream velocity for design event – 16 feet per second
- Estimated scour depth – 5.5 feet
- Length of bank requiring protection – 550 feet
- Bank slope – 1.5:1
- Specific gravity of rock used for RSP – 2.65 (based on data from local quarry)
- Embankment is on outside of stream bend

1) Calculate minimum rock mass for outer layer:

$$W = \frac{(0.00002)\left(16 \times 4/3\right)^6 (2.65)}{(2.65-1)^3 \sin^3(70-33.69)}$$

W = 5,350 lb

W = 2.67 ton = 2.43 tonne

NOTES:

For ease of computation with hand held calculators, cosecant has been converted to 1/sine.)

2) Select gradation for outer layer.

a) From minimum calculated rock weight of 2.67 tons in the example, select the rock weight from the left-side column tables in Standard Specification Section 72-2.02 that represents the standard rock weight just larger than the calculated weight. For ease, the Standard Specification tables are combined and reprinted in Table 873.3A.

   The next larger rock mass above 2.67 ton is 4 ton. RSP this large is only to be installed using Method A placement techniques (i.e., individual rock placement, no end dumping). From this value, move horizontally across the gradation ranges to the “50-100” entry. From here, move vertically upward to select the design gradation, or RSP Class.

   In this instance the name of the RSP class is 4 T.

b) Generally, this will represent the design outer RSP layer. However, the designer must assess this value against the site conditions observed during the field review and in conjunction with site history and projected future conditions prior to finalizing the selection. For the purposes of this example, we will assume this design gradation (i.e., 4 T RSP class) is appropriate.

Table 873.3A
Guide for Determining RSP-Class of Outside Layer

Standard Rock Sizes	GRADING OF ROCK SLOPE PROTECTION PERCENTAGE LARGER THAN											
	RSP-Classes [1]											
	Method A Placement					Method B Placement						
	RSP-Classes other than Backing									Backing No.		
	8 T	4 T	2 T	1 T	1/2 T	1 T	1/2 T	1/4 T	Light	1	2	3
16 ton	0-5											
8 ton	50-100	0-5										
4 ton	95-100	50-100	0-5									
2 ton		95-100	50-100	0-5		0-5						
1 ton			95-100	50-100	0-5	50-100	0-5					
1/2 ton				95-100	50-100	-----	50-100	0-5				
1/4 ton					95-100	95-100	-----	50-100	0-5			
200 lb							95-100	-----	50-100	0-5		
75 lb								95-100	-----	50-100	0-5	
25 lb									95-100	90-100	25-75	0-5
5 lb											90-100	25-75
1 lb												90-100

[1] “Facing” has same gradation as “Backing No. 1”. To conserve space “Facing” is not shown.

3) Determine RSP Layers. As previously discussed, properly designed RSP revetments are comprised of multiple layers of progressively smaller rock gradations progressing from the large sized rocks of the outer layer to the native soil or constructed embankment. Where the outer layer is composed of relatively small rock only a single inner layer may be needed. For a large rock outer layer as many as three inner layers may be required.

   For this example, the outer RSP layer is 4 T. From Table 873.3B, there are two options for the inner layers. The reason for multiple options for the larger RSP gradation classes is to allow the designer to better select RSP that is available from local quarry sources. Either set of layered designs is acceptable. The designer should contact rock producers in proximity to the project site to obtain price quotes for the different alternatives.

   This information may also be available from the District Materials Engineer. For the purposes of this example, we will select the layered design of: 4 T, 1 T, ¼ T, Backing No. 2 and Class 10 RSP Fabric.

4) Determine Thickness of Revetment. RSP layers are composed of rock classes shown in Table 873.3A. Each layer is at least 1.5 times the diameter of the median sized rock (D_{50}) in the gradation in order to prevent the smaller rocks in the lower layers from migrating.

   Table 873.3C provides the required thickness for the various RSP gradations and types of placement (Method A or Method B). Method B placement requires an increase in thickness to account for the looser rock contact and difficulty in controlling layer thickness inherent in end dumping of rock.

   Based on the table values, the total thickness of the design in our example (measured normal to the slope) is:

   4 T Layer = 6.8 ft

   1 T Layer = 4.3 ft

   ¼ T Layer = 3.3 ft

   Backing No. 2 Layer = 1.25 ft

   RSP Fabric = Effectively

   \+ 0.0 ft

   Total = 15.35 ft

5) Assess Stream Impact Due to Revetment. In some cases, the thickness of the completed RSP revetment creates a narrowing of the available stream channel width, to the extent that stream velocity or stage at the design event is increased to undesirable levels, or the opposite bank becomes susceptible to attack. In these cases, the bank upon which the RSP is to be placed must be excavated such that the constructed face of the revetment is flush with the original embankment.

6) Exterior Edges of Revetment. The completed design must be compatible with existing and future conditions. Freeboard and top edge of revetments were covered in

Table 873.3B

California Layered RSP

Outsider Layer RSP-Class *	Inner Layers RSP-Class *	Backing Class No. *	RSP-Fabric Class **
8 T	2 T over ½ T	1	10
8 T	1 T over ¼ T	1 or 2	10
4 T	½ T	1	10
4 T	1 T over ¼ T	1 or 2	10
2 T	½ T	1	10
2 T	¼ T	1 or 2	10
1 T	Light	None	8
1 T	¼ T	1 or 2	8
½ T	None	1	8
¼ T	None	1 or 2	8
Light	None	None	8
Backing No.1 ***	None	None	8

NOTES:

* Rock grading and quality requirements per Standard Specifications.

** RSP-fabric Type of geotextile and quality requirements per Section 88 Rock Slope Protection Fabric of the Standard Specifications. Class 8 RSP-fabric has lower weight per unit area and it also has lower toughness (tensile x elongation, both at break) than Class 10 RSP-fabric.

*** "Facing" RSP-Class has same gradation as Backing No. 1.

Table 873.3 C

Minimum Layer Thickness

RSP-Class Layer	Method of Placement	Minimum Thickness
8 T	A	8.5 ft
4 T	A	6.8 ft
2 T	A	5.4 ft
1 T	A	4.3 ft
½ T	A	3.4 ft
1 T	B	5.4 ft
½ T	B	4.3 ft
¼ T	B	3.3 ft
Light	B	2.5 ft
Facing	B	1.8 ft
Backing No. 1	B	1.8 ft
Backing No. 2	B	1.25 ft
Backing No. 3	B	0.75 ft

Index 873.3(2)(a)(2)(b) "Design Height." For depth of toe, the estimated scour was given as 5.5 feet. This is the minimum toe depth to be considered. Again, based on site conditions and discussions with maintenance staff and others, determine if any long-term conditions need to be addressed. These could include streambed degradation due to local aggregate mining or headcutting. Regardless of the condition, the toe must be founded below the lowest anticipated elevation that could become exposed overthe service life of the embankment or roadway facility. As for the upstream and downstream ends, the given length of revetment is 500 feet. Again, this will

typically be a minimum, as the designer should seek natural rock outcroppings, areas of quiescent stream flow, or other inherently stable bank segments to end the RSP, see Figure 873.3D for example at ocean shore location.

Figure 873.3D
RSP Lined Ocean Shore

RSP placed at site subject to deep water wave attack. Terminal end of RSP tied into natural rock outcropping.

(b) Rock Slope Shore Protection.

(1) General Features. Rock slope protection when used for shore protection, in addition to the general advantages listed previously for streambank rock slope protection, reduces wave runup as compared to smooth types of protection.

(a) Method A placement is normally specified for ocean shore protection since very large stone is typically needed. Rock mass for lake shores and protected bays are often based on the height of boat generated waves.

(b) Foundation treatment in shore protection may be controlled by tidal action as well as excavation difficulties and production may be limited to only two or three toe or foundation rocks per tide cycle. If toe rocks are not properly bedded, the subsequent vertical adjustment may be detrimental to the protection above. Even though rock is self-adjusting, the bearing of one rock to another may be lost. It is often necessary to construct the toe or foundation to an elevation approximating high tide in advance of embankment construction to prevent erosion of the embankment.

(2) Shore Protection Design.

(a) Stone Size -- For waves that are shoaling as they approach the protection the required stone size may be determined by Using Chart B, Figure 873.3G.

The nomograph is derived from the following formula:

$$W = \frac{0.003 d_B{}^3 sg_r \csc^3(\beta - \alpha)}{\left(\frac{sg_r}{sg_w} - 1\right)^3}$$

Where:

d_B = maximum depth in feet of water at toe of the rock slope protection, see Figure 873.3C

sg_r = specific gravity of stones

sg_w = specific gravity of water (sea water = 1.0265)

α = angle of face slope from the horizontal

β = 70 for broken rock, a constant

W = weight of minimum stable stone in lbs

In general, d_B will be the difference between the elevation of the scour line at the toe and the maximum stillwater level. For ocean shore, d_s may be taken as the distance from the scour line to mean sea level plus one-half the maximum tidal range.

If the deep-water waves, see Figure 873.3D, reach the protection, the stone size may be determined by using Chart A, Figure 873.3G. The nomograph is derived from the following formula:

$$W = \frac{0.00231 H_d^{\,3} sg_r \csc^3(\beta - \alpha)}{\left(\frac{sg_r}{sg_w} - 1\right)^3}$$

Where:

H_d = design wave in feet, see Index 873.2

If in doubt whether waves generated by fetch and wind velocity will be of sufficient size to be affected by shoaling, use both charts and adopt the smaller value.

(b) Dimensions -- Rock should be founded in a toe trench dug to hard rock or keyed into soft rock. If bedrock is not within reach, the toe should be carried below the estimated depth of probable scour. If the scour depth is questionable, additional thickness of rock may be placed at the toe which will adjust and provide deeper support. In determining the elevation of the scoured beach line the designer should observe conditions during the winter season, consult records, or ask persons who have a knowledge of past conditions.

Wave run-up is reduced by the rough surface of rock slope protection. In order that the wash will not top the rock, it should be carried up to an elevation of twice the maximum depth of water ($2d_s$) or to an elevation equal to the maximum depth of water plus the deep-water wave height ($d_s + H_d$), whichever is the *lower*. See Figure 873.3C.

Consideration should also be given to protecting the bank above the rock slope protection from splash and spray.

Design thickness of the protection should be based on the same procedures as used for streambanks. For typical conditions the thickness required for the various sizes are shown on Table 873.3B. Except for toes on questionable foundation, as explained above, additional thickness will not compensate for undersized stones. When properly constructed, the largest stones will be on the outside, and if the wave forces displace these, additional thickness will only add slightly to the time of failure. Shore revetments, particularly ocean shore locations, are often candidates for using a mounded toe design. Where it is not practical to excavate to bedrock or to the anticipated scour depth to set the revetment toe, an alternative treatment is to place additional rock (i.e., mound) of the same mass as the outer layer at the toe. The volume to be placed should be slightly greater than the amount that would have been needed to extend the toe to the estimated scour depth. See figure 873.3C for a depiction of a mounded toe installation.

As scour occurs at the toe of the revetment, this mounded rock will drop into the scour hole. It is important in mounded toe designs to require that excess RSP fabric be placed so that as the scour hole develops and rock begins to drop, the excess RSP fabric will “unfold” and also drop into place to limit loss of embankment.

(c) Gabions. Gabion revetments consist of rectangular wire mesh baskets filled with stone. See Standard Plan D100A and D100B for gabion basket details and the Standard Specifications for requirements.

Gabions are formed by filling commercially fabricated and preassembled wire baskets with rock. There are two types of gabions, wall type and mattress type. In wall type the empty cells are positioned and filled in place to form walls in a stepped fashion. Mattress type baskets are positioned on the slope and filled. Wall type revetment is not fully self adjusting but has some flexibility. The mattress type is very flexible. For some locations, gabions may be more aesthetically acceptable than rock riprap. Where larger stone sizes are not readily available and the flow does not abrade the wire baskets, they may also be more cost effective. However, caution is advised regarding in-stream placement of gabions, and some form of abrasion protection in the form of wooden planks or other facing will typically be necessary, see Figure 873.3E.

(d) Articulated Precast Concrete. This type of revetment consists of pre-cast concrete blocks which interlock with each other, are attached to each other, or butted together to form a continuous blanket or mat. A number of block designs are commercially available. They differ in shape and method of articulation, but share common features of flexibility and rapid installation. Most provide for establishment of vegetation within the revetment.

The permeable nature of these revetments permits free draining of the embankment and their flexibility allows the mat to adjust to minor changes in bank geometry. Pre-cast concrete block revetments may be economically justified where suitable rock for slope protection is not readily available. They are generally more aesthetically pleasing than other types of revetment, particularly after vegetation has become established.

Individual blocks are commonly joined together with steel cable or synthetic rope, to form articulated block mattresses. Pre-assembled in sections to fit the site, the mattresses can be used on slopes up to 2:1. They are anchored at the top of the revetment to secure the system against slippage.

Pre-cast block revetments that are formed by butting individual blocks end to end, with no physical connection, should not be used on slopes steeper than 3:1. An engineering fabric is normally used on the slope to prevent the erosion of the underlying embankment

Figure 873.3E

Gabion Lined Streambank

Gabion wall with timber facing to protect wires from abrasive flow.

through the voids in the concrete blocks.

Refer to HEC-11, Design of Riprap Revetment, Section 6.2, and HEC-23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 4, for further discussion on the use of articulated concrete blocks.

(3) *Rigid Revetments.*

(a) Concreted-Rock Slope Protection.

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

It has application in areas where rock of sufficient size for ordinary rock slope protection is not economically available.

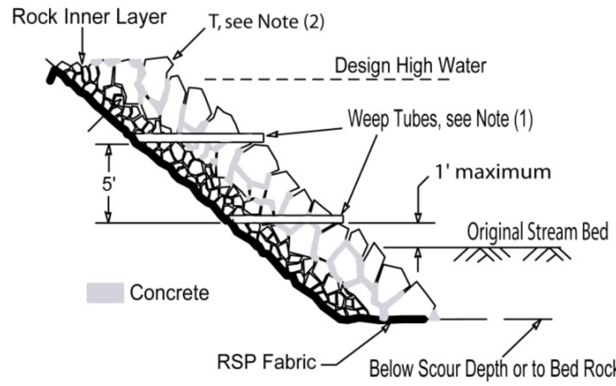
(2) Design Concepts. Concreting of RSP is a common practice where availability of large stones is limited, or where there is a need to reduce the total thickness of a RSP revetment. Inclusion of the concrete, and the labor required to place it, makes concreted RSP installations more expensive per unit area than non-concreted installations.

Design procedures for concreted RSP revetments are similar to that of non-concreted RSP. Start by following the design example provided in Index 873.3(2)(a)(2)(c) to select a stable rock size for a non-concreted design based on the site conditions. This non-concreted rock size is divided by a factor of roughly four or five to arrive at the appropriate size outer layer rock for a concreted revetment. The factor is based on observations of previously constructed facilities and represents the typical sized pieces that stay together even after severe cracking (i.e., failed revetments will still usually have segments of four to five rocks holding together). As with the non-concreted design procedures, use the rock size derived from this calculation to enter Table 873.3A (i.e., round up to the next larger rock mass, which will represent the 50-100 percentage larger than gradation range) and then select the appropriate RSP Class. The thickness and rock sizing of the inner layers can be based on the reduced sizing of the outer layer rock. Note that as shown in Figure 873.3F, the inner layers of rock are not concreted.

As this type of protection is rigid without high strength, support by the embankment must be maintained. Slopes steeper than the angle of repose of the embankment are risky, but with rocks grouted in place, little is to be gained with slopes flatter than 1.5:1. Precautions to prevent undermining of embankment are particularly important, see Figure 873.3H. The concreted-rock must be founded on solid rock or below the depth of possible scour. Ends should be protected by tying into stable rock or

Figure 873.3F

Concreted-Rock Slope Protection



NOTES:

(1) If needed to relieve hydrostatic pressure.

(2) Refer to Table 873.3 C for section thickness.

Dimensions and details should be modified as required.

forming smooth transitions with embankment subjected to lower velocities. As a precaution, cutoff stubs may be provided. If the embankment material is exposed at the top, freeboard is warranted to prevent overtopping.

Figure 873.3H
Toe Failure - Concreted RSP

Toe of concreted RSP that has been undermined.

The design intent is to place an adequate volume of concrete to tie the rock mass together, but leave the outer face roughened with enough rock projecting above the concrete to slow flow velocities to more closely approximate natural conditions.

The volume of concrete required is based on filling roughly two-thirds of the void space of the outer rock layer, as shown in Figure 873.3F. The concrete is rodded or vibrated into place leaving the outer stones partially exposed. Void space for the various RSP gradations ranges from approximately 30 percent to 35 percent for Method A placed rock to 40 percent to 45 percent for Method B placed rock of the total volume placed.

(2) Specifications. Quality specifications for rock used in concreted-rock slope protection are usually the same as for rock used in ordinary rock slope protection. However, as the rocks are protected by the concrete which surrounds them, specifications for specific gravity and hardness may be lowered if necessary. The concrete used to fill the voids is normally 1 inch maximum size aggregate minor concrete. Except for freeze-thaw testing of aggregates, which may be waived in the contract special provisions, the concrete should conform to the provisions of Standard Specification Section 90.

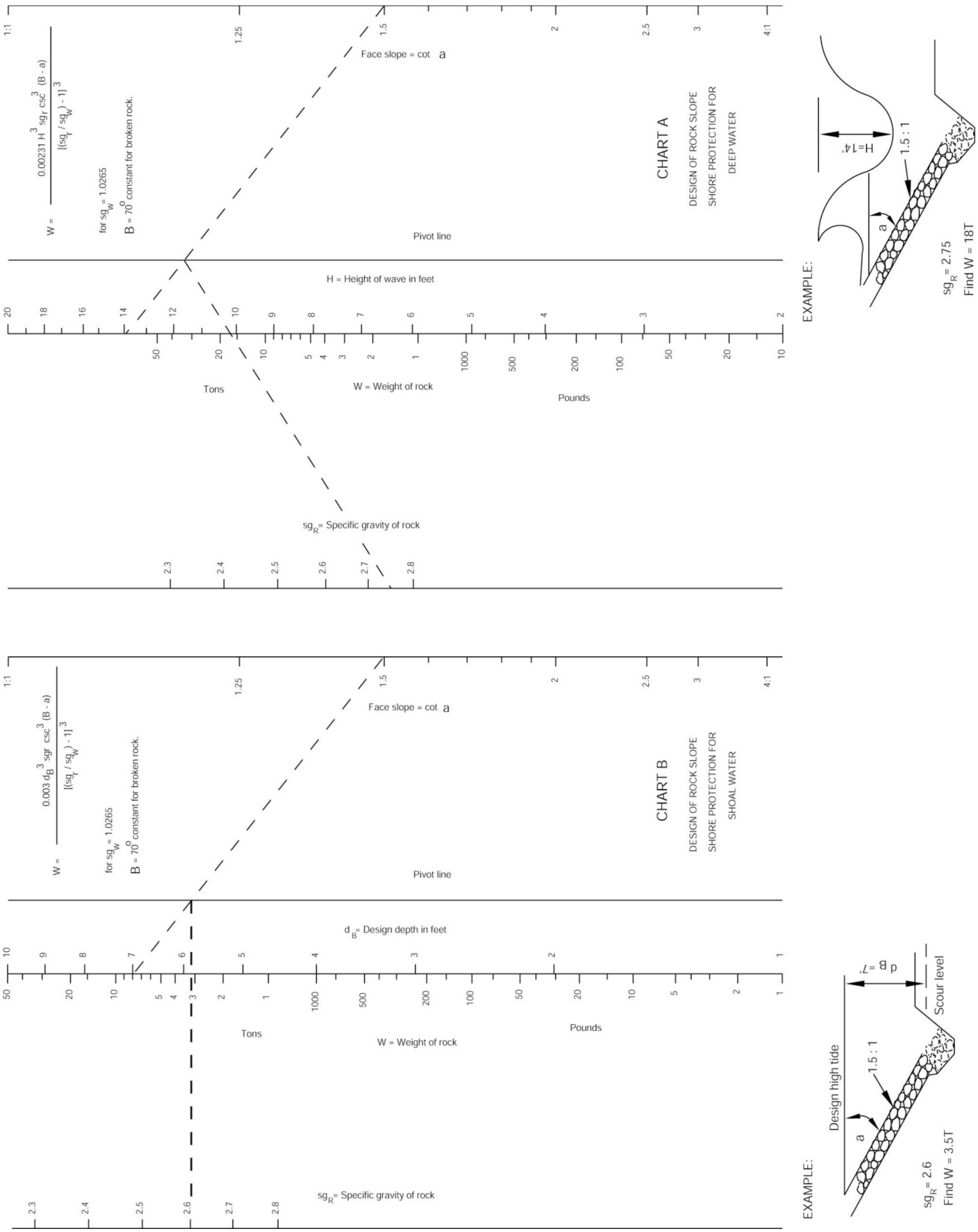
Size and grading of stone and concrete penetration depth are provided in Standard Specification Section 72.

(b) Sacked-Concrete Slope Protection. This method of protection consists of facing the embankment with sacks filled with concrete. It is expensive, but historically was a much used type of revetment. Much hand labor is required but it is simple to construct and adaptable to almost any embankment contour. Use of this method of slope protection is generally limited to replacement or repair of existing sacked concrete facilities, or for small, unique situations that lend themselves to hand-placed materials.

Tensile strength is low and as there is no flexibility, the installation must depend almost entirely upon the stability of the embankment for support and therefore should not be placed on face slopes much steeper than the angle of repose of the embankment material. Slopes steeper than 1:1 are rare; 1.5:1 is common. The flatter the slope, the less is the area of bond between sacks. From a construction standpoint it is not practical to increase the area of bond between sacks; therefore for slopes as flat as 2:1 all sacks should be laid as headers rather than stretchers.

Integrity of the revetment can be increased by embedding dowels in adjoining sacks to reinforce intersack bond. A No. 3 deformed bar driven through a top sack into the underlying sack while the concrete is still fresh is effective. At cold joints, the first course of sacks should be impaled on projecting bars that were driven into the last previously placed course. The extra

Figure 873.3G
Nomographs For Design of Rock Slope Shore Protection



strength may only be needed at the perimeter of the revetment.

Most failures of sacked concrete are a result of stream water eroding the embankment material from the bottom, the ends, or the top.

The bottom should be founded on bedrock or below the depth of possible scour.

If the ends are not tied into rock or other nonerosive material, cutoff returns are to be provided and if the protection is long, cutoff stubs are built at 30-foot intervals, in order to prevent or retard a progressive failure.

Protection should be high enough to preclude overtopping. If the roadway grade is subject to flooding and the shoulder material does not contain sufficient rock to prevent erosion from the top, then pavement should be carried over the top of the slope protection in order to prevent water entering from this direction.

Class 8 RSP fabric as described in Standard Specification Section 88 should be placed behind all sacked concrete revetments. For revetments over 4 feet in height, weep tubes should also be placed, see Figure 873.3F.

For good appearance, it is essential that the sacks be placed in horizontal courses. If the foundation is irregular, corrective work such as placement of entrenched concrete or sacked concrete is necessary to level up the foundation. Refer to HDS No. 6, Section 6.6.5, for further discussion on the use of sacked concrete slope protection.

(c) Concrete Slope Paving.

(1) General Features. This method of protection consists of paving the embankment with portland cement concrete. Slope paving is used only where flow is controlled and will not over-top the protection.

It is particularly adaptable to locations where high-velocity flow is not detrimental but desirable and the hydraulic efficiency of smooth surfaces is important. It has been used very little in shore protection. On a cubic feet basis the cost is high but as the thickness is generally only 3 inches to 6 inches, the cost on a basis of area covered will usually be less than for sacked-concrete slope protection. This is especially so when sufficiently large quantities are involved and alignment is such as to warrant the use of mass production equipment such as slip-form pavers.

Due to the rigidity of PCC slope paving, its foundation must be good and the embankment stable. Although reinforcement will enable it to bridge small settlements of the embankment face, even moderate movements could lead to cracking of the paving or failure. The toe must be on bedrock or extend below possible scour. When this is not feasible without costly underwater construction, rock or PCC grouted RSP have been used as a foundation. A better but much more expensive solution is to place the toe on a PCC wall or piles.

Every precaution must be taken to exclude stream water from pervious zones behind the slope paving. The light slabs will be lifted by comparatively small hydrostatic pressures, opening joints or cracks at other points in a series of progressive failures leading to extensive or complete failure.

Considering the severity of failure from bank erosion or hydrostatic pressure after overtopping, 1 foot to 2 feet of freeboard above design high water is recommended for this type of revetment. Refer to HEC-11, Design of Riprap Revetment, Section 6.4, for further discussion on the use of concrete slope paving. Table 873.3D gives channel lining thickness.

(4) Bulkheads. A bulkhead is a steep or vertical structure supporting a natural slope or

constructed embankment. As bank and shore protection structures, bulkheads serve to secure the bank against erosion as well as retaining it against sliding. As a slope protection structure, revetment design principles are used, the only essential difference being the steepness of the face slope. As a retaining structure, conventional design methods for retaining walls, cribs and laterally loaded piles are used.

Bulkheads are usually expensive, but may be economically justified in special cases where valuable riparian property or improvements are involved and foundation conditions are not satisfactory for less expensive types of slope protection. They may be used for toe protection in combination with other revetment types of slope protection. Some other considerations that may justify the use of bulkheads include:

- Encroachment on a channel cannot be tolerated.
- Retreat of highway alignment is not viable.
- Right of Way is restricted.
- The force and direction of the stream can best be redirected by a vertical structure.

The foundation for bulkheads must be positive and all terminals secure against erosive forces. The length of the structure should be the minimum necessary, with transitions to other less expensive types of slope protection when possible. Eddy currents can be extremely damaging at the terminals and transitions. If overtopping of the bulkheads is anticipated, suitable protection should be provided.

Along a stream bank, using a bulkhead presumes a channel section so constricted as to prohibit use of a cheaper device on a natural slope. Velocity will be unnaturally high along the face of the bulkhead, which must have a fairly smooth surface to avoid compounding the restriction. The high velocity will increase the threat of scour at the toe and erosion at the downstream end. Allowance must be made for these threats in selecting the type of foundation, grade of footing, penetration of piling, transition, and anchorage at downstream end. Transitions at both ends may appropriately taper the width of channel and slope of the bank. Transition in roughness is desirable if attainable. Refer to HDS No. 6, Section 6.4.8, for further discussion on the use of bulkheads to prevent streambank erosion or failure.

Along a shore, use of a bulkhead presumes a steep lake or sea bed profile, such that revetment on a 1.5:1 or flatter slope would project into prohibitively deep water or permit intolerable wave runup. Such shores are generally rocky, offering good foundation on residual reefs, but historic destruction of the overlying formation attests to the hydraulic power of the sea to be resisted by an artificial replacement. The face of such a bulkhead must be designed to absorb or dissipate as much as practical the shock of these forces. Designers should consult the U.S. Army Corps of Engineers EM-1110-2-1614, Design of Coastal Revetments, Seawalls, and Bulkheads, for more complete information and details.

(a) Concrete or Masonry Walls. The expertise and coordination of several engineering disciplines is required to accomplish the development of PS&E for concrete walls serving the dual purpose of slope protection and support. The Division of Structures is responsible for the structural integrity of all retaining walls, including bulkheads.

(b) Crib walls. Timber and concrete cribs can be used for bulkheads in locations where some flexibility is desirable or permissible. Metal cribs are limited to support of embankment and are not recommended for use as protection because of vulnerability to corrosion and abrasion.

The design of crib walls is essentially a determination of line, foundation grade, and height with special attention given to potential scour and possible loss of backfill at the base and along the toe. Design details for concrete crib walls are shown on Standard Plans C7A through C7G. Concrete crib walls used as bulkheads and exposed to salt water require special provisions specifying the use of coated rebars and special high density concrete.

Recommendations from METS Corrosion Technology Branch should be requested.

Design details for timber crib walls of dimensioned lumber are shown on Standard Plans C9A and C9B. Timber cribs of logs, notched to interlock at the contacts, may also be used. All dimensioned lumber should be treated to resist decay.

(c) Sheet Piling. Timber, concrete and steel sheet piling are used for bulkheads that depend on deep penetration of foundation materials for all or part of their stability. High bulkheads are usually counterforted at upper levels with batter piles or tie back systems to deadmen. Any of the three materials is adaptable to sheet piling or a sheathed system of post or column piles.

Excluding structural requirements, design of pile bulkheads is essentially as follows:

- Recognition of foundation conditions suitable to or demanding deep penetration. Penetration of at least 15 feet below scour level, or into soft rock, should be assured.
- Choice of material. Timber is suitable for very dry or very wet climates, for other situations economic comparison of preliminary designs and alternative materials should be made.
- Determination of line and grade. Fairly smooth transitions with protection to high-water level should be provided.

(5) Vegetation. Vegetation is the most natural method for stabilization of embankments and channel bank protection. Vegetation can be relatively easy to maintain, visually attractive and environmentally desirable. The root system forms a binding network that helps hold the soil. Grass and woody plants above ground provide resistance to the near bank water flow causing it to lose some of its erosive energy.

Erosion control and revegetation mats are flexible three-dimensional mats or nets of natural or synthetic material that protect soil and seeds against water erosion prior to establishment of vegetation. They permit vegetation growth through the web of the mat material and have been used as temporary channel linings where ordinary seeding and mulching techniques will not withstand erosive flow velocities. The designer should recognize that flow velocity estimates and a particular soils resistance to erosion are parameters that must be based on specific site conditions. Using arbitrarily selected values for design of vegetative slope protection without consultation with the District Hydraulic Unit and/or the District Landscape Architect Unit is not recommended. However, a suggested starting point of reference is Table 865.2 in which the resistance of various unprotected soil classifications to flow velocities are given. Under near ideal conditions, ordinary seeding and mulching methods cannot reasonably be expected to withstand sustained flow velocities above 4 feet per second. If velocities are in excess of 4 feet per second, a lining maybe needed, see Table 873.3E.

Temporary channel liners are used to establish vegetative growth in a drainage way or as slope protection prior to the placement of a permanent armoring. Some typical temporary channel liners are:

- Straw
- Excelsior
- Jute
- Woven paper

Vegetative and temporary channel liners are suitable for conditions of uniform flow and moderate shear stresses.

Permanent soil reinforcing mats and rock riprap may serve the dual purpose of temporary and permanent channel liner. Some typical permanent channel liners are:

- Gravel or cobble size riprap
- Fiberglass roving
- Geosynthetic mats
- Polyethelene cells or grids
- Gabion Mattresses

Table 873.3E

Permissible Velocities for Flexible Channel Linings

Type of Lining [1]	Permissible Velocity (ft/s)	
	Intermittent Flow	Sustained Flow
Vegetation:		
Bermuda Grass, uncut	4.0	2.5
Bermuda Grass, mowed or Crab Grass, uncut	4.0	2.5
Riprap:		
Gravel, 1 in	3.0	2.0
Gravel, 2 in	3.5	2.5
Cobble, 3 in	5.0	4.0
Cobble, 6 in	7.5	6.5
Temporary:		
Woven Paper Net	4.5	3.5
Jute Net	5.0	4.0
Fiberglass Roving	5.5	4.5
Straw with Net	6.5	4.5
Curled Wood Mat	6.5	4.5
Synthetic Mat	10.5	7.5

NOTE:

(1) Ref. HEC-15.

However, geosynthetics and plastic (polyethylene, polypropylene, polyamide, etc.) based mats must be installed in a fashion where there will be no potential for long-term sunlight exposure, as these products will degrade due to UV radiation.

Composite designs are often used where there are sustained low flows of high to moderate velocities and intermediate high water flows of low to moderate velocities. Brush layering is a permanent type of erosion control technique that may also have application for channel protection, particularly as a composite design.

Additional design information on vegetation, and temporary and permanent channel liners is given in Chapter IV, HEC-15, Design of Roadside Channels and Flexible Linings.

873.4 Training Systems

(1) General. Training systems are structures, usually within a channel, that act as countermeasures to control the direction, velocity, or depth of flowing water. As shore protection, they control shoaling and scour by deflecting the strength of currents and waves.

The degree of permeability is among the most important properties of control structures. An impermeable structure may deflect a current entirely, whereas a permeable structure may serve mainly to reduce the strength of water velocity, currents or waves.

Training systems of the retard and permeable jetty types are similar in that they are usually extensive or multi-unit open structures like; piling, fencing, and unit frames. They are dissimilar in function and alignment, retards being parallel and groins oblique to the banks. The retard is a milder remedy than jetty construction.

(a) Retard Types. A retard is a bank protection structure designed to check riparian velocity and induce silting and accretion. They are usually placed parallel to the highway embankment or erodible banks of channels on stable gradients. Retards typically take the following forms of construction:

- Fencing - single or double lines
- Palisades - piles and netting
- Timber piling or pile bents
- Steel or timber jacks

Retards are applicable primarily on streams which meander to some extent within a mature valley. Typical uses include the following:

- Protection at the toe of highway embankments that encroach on a stream channel.
- Training and control to inhibit erosion upstream and downstream from stream crossings.
- Control of erosion redeposition of material where progressive embayments are creating a problem.

(1) Fence Type. Fence-type structures are used as retards, permeable or impermeable jetties, and as baffles. These structures can be constructed of various materials.

Fence type retards may be effective on smaller streams and areas subject to infrequent attack, such as overflow areas. Single and double rows of various types of fencing have been used. The principal difference between fence retards and ordinary wire fences is that the posts of retards must be driven sufficiently deep to avoid loss by scour.

Permeability can be varied in the design to fit the requirements of the location for single fences, the factor most readily varied is the pattern of the wire mesh. For multiple fences, the mesh pattern can be varied or the space between fences can be filled to any desired height. Making optimum use of local materials, this fill may be brush ballasted by rock, or rock alone.

(2) Piles and Palisades. Retards and jetties may be of single, double, or triple rows of piles with the outside or upstream row faced with wire mesh fencing material, boards or polymeric straps

interwoven into a high-strength net. The facing adds to the retarding effect and may trap light brush or debris to supplement its purpose. This type retard is particularly adapted to larger streams where the piles will remain in the water. The number of pile rows and amount of facing may be varied to control the deposition of material. In leveed rivers it is often desirable to discourage accretion so as to not constrict the channel but provide sufficient retarding effect to prevent loss of a light bank protection such as vegetation or light rock facing.

Typical design considerations include:

- If the stream carries heavy debris, the elevation of the top of the pile should be well below the high-water level in order that heavy objects such as logs will pass over the top during normal floods.
- Piles must have sufficient penetration to prevent loss from scour or impact by floating debris or both. This is especially important for the piles at the outer end of jetties. If scour is a problem, the pile may be protected by a layer of rock placed on the streambed. Piles should be long enough to penetrate below probable scour, with penetration of a least 15 feet in streams with sandy beds and velocities of 10 feet per second to 15 feet per second.
- Ends of the system should be joined to the bank in order to prevent parallel high-velocity flow between the retard and the bank. If the installation is long, additional bank connections may be placed at intervals.
- Facing material should be fastened to the upstream or channel side of the piling in order that the force of the water and impact of debris will not be entirely on the fasteners.

(3) Jacks and Tetrahedrons. Jacks and tetrahedrons are skeletal frames that can be used as retards or permeable jetties. Cables can be used to tie a number of similar units together in longitudinal alignment and for anchorage of key units to deadmen. Struts and wires are added to the basic frames to increase impedance to flow of water directly by their own resistance and indirectly by the debris they collect.

Both devices serve best in meandering streams which carry considerable bed load during flood stages. Impedance of the stream along the string of units will cause deposit of alluvium, especially at the crest and during the falling stage. Beds of such streams often scour on the rising stage, undercutting the units and causing their subsidence, often accompanied by rotation when one leg or side is undercut more than the other. Deposition of the falling stage usually restores the former bed, partially or completely burying the units. In that lowered and rotated position, they may still be completely effective in future floods.

Retards may be used alone or in combination with other types of slope protection. In combination with a lighter type of armor they may be more economical than a heavier type of protection. They can be used as toe protection for other types of slope protection where a good foundation is impractical because of high water or extreme depth of poor material.

Where new embankment is placed behind the retard consideration should be given to protecting the slope to inhibit erosion until the retard has had an opportunity to function. The slope protection used should promote the establishment of a natural cover, such as discussed under Index 873.3(5), Vegetation.

Retards on tangent reaches of narrow channels may, by slowing the velocity on one side, cause an increase in velocity, on the other. On wider reaches of a meandering stream they may, by slowing a rebounding high velocity thread, have a beneficial effect on the opposite bank. Where the prime purpose of the retard system is to reduce stream bank velocity to encourage deposition of material intended to alter the channel alignment the effect on adjacent property must be assessed. Where deposition of material is the primary function, the service life of the installation is dependent on the deposition rate and the ultimate establishment of a natural retard.

The length of a retard system should extend from a secure anchorage on the upstream end to anchorage on the downstream end beyond the area under direct attack. Since erosion often progresses downstream, this possibility should be considered in determining the planned length.

The top of a retard need not extend to the elevation of design high water. In major rivers and streams where drift is large and heavy it is essential that the retard be low enough to pass debris over the top during stages of high flow.

For further information on retards, refer to Section 6.4.4 of HDS No. 6.

(b) Jetty Types. A jetty is an elongated artificial obstruction projecting into a stream or the sea from bank or shore to control shoaling and scour by deflection or redirection of currents and waves. When used in stream environments, a common term used for these devices is spur dike.

This classification may be subdivided with respect to permeability. Impermeable jetties being used to deflect the stream and permeable jetties being used not only to deflect the stream but to permit some flow through the structure to minimize the formation of eddies immediately downstream. Most jetty installations are permeable structures.

Permeable jetties typically take the following forms of construction:

- Palisades -- piles and netting.
- Single and double rows of timber-braced piling.
- Steel or timber jacks.
- Precast concrete, interlocking shapes or hollow blocks.

Impermeable jetties typically take the following forms of construction:

- Guide and spur dikes, earth or rock.
- PCC grouted riprap dikes.
- Single and double lines of sheeting or sheet piling (steel, timber or concrete, framed and braced or on piling).
- Double fence, filled.
- Log or timber cribs, filled.

Impermeable jetties in the form of filled fences and cribs have been used with only limited success. Characteristic performance of these is the development of an eddy current immediately downstream which attacks the bank and often requires secondary protective measures.

Basic principles for permeable jetties are much the same as for retards, the important difference being that they deflect the flow in addition to encouraging deposition. The preceding comment on retards should be considered as related and applicable to jetties when qualified by this basic difference.

Permeable jetties are placed at an angle with the embankment and are more applicable in meandering streams for the purpose of directing or forcing the current away from the embankment, see Figure 873.4A. When the purpose is to deposit material and promote growth, the jetties are considered to have fulfilled their function and are expendable when this occurs.

Figure 873.4A

Thalweg Redirection Using Bendway Weirs

Bendway weirs in conjunction with rock slope protection.

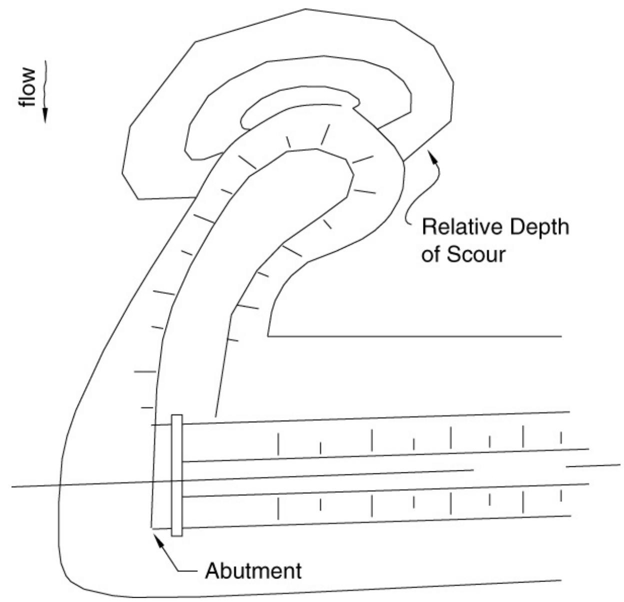
They also encourage deposition of bed material and growth of vegetation. Retards build a narrow strip in front of the embankment, where as permeable jetties cover a wider area roughly limited by the envelope of the outer ends.

The relation between length and spacing of jetties should approximate unity as a general rule to assure complete entrapment and retention of material. The spacing can be increased if the resulting scalloped effect is not detrimental to the desired result. See HEC-23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 9 for additional information.

(c) Guide Dikes/Banks. Guide banks are appendages to the highway embankment at bridge abutments, see Figure 873.4B. They are smooth extensions of the fill slope on the upstream side. Approach embankments are frequently planned to project into wide floodplains, to attain an economic length of bridge. At these locations high water flows can cause damaging eddy currents that scour away abutment foundations and erode approach embankments. The purpose of guide dikes is twofold. The first is to align flow from a wide floodplain toward the bridge opening. The second is to move the damaging eddy currents from the approach roadway embankment to the upstream end of the dike.

Figure 873.4B

Bridge Abutment Guide Banks



Guide banks are usually earthen embankment faced with rock slope protection. Optimum shape and length of guide dikes will be different for each site. Field experience has shown that an elliptical shape with a major to minor axis ratio of 2.5:1 is effective in reducing turbulence. The length is dependent on the ratio of flow diverted from the floodplain to flow in the first 100 feet of waterway under the bridge. If the use of another shape dike, such as a straight dike, is required for practical reasons more scour should be expected at the upstream end of the dike. The bridge end will generally not be immediately threatened should a failure occur at the upstream end of a guide dike.

Toe dikes are sometimes needed downstream of the bridge end to guide flow away from the structure so that redistribution in the floodplain will not cause erosion damage to the embankment due to eddy currents. The shape of toe dikes is of less importance than it is with upstream guide banks.

For further information on spur dike and guide bank design procedures, refer to Section 6.4 of HDS No. 6. General design considerations and guidance for evaluating scour and stream stability at highway bridges is contained in HEC-18, HEC-20, and HEC-23.

(d) Groins. A groin is a relatively slender barrier structure usually aligned to the primary motion of water designed to trap littoral drift, retard bank or shore erosion, or control movement of bed load.

These devices are usually solid; however, upon occasion to control the elevation of sediments they may be constructed with openings. Groins typically take the following forms of construction:

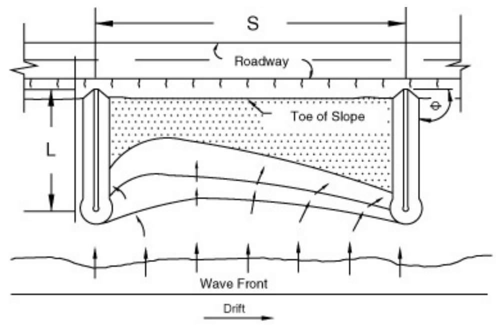
- Rock mound.
- Concreted-rock dike.
- Sand filled plastic coated nylon bags.
- Single or double lines of sheet piling.

The primary use of groins is for ocean shore protection. When used as stream channel protection to retard bank erosion and to control the movement of streambed material they are normally of lighter construction than that required for shore installation.

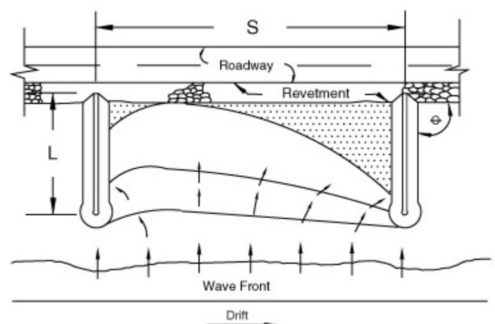
In its simplest or basic form, a groin is a spur structure extending outward from the shore over beach and shoal. A typical layout of a shore protection groin installation is shown in Figure 873.4C.

Assistance from the U.S. Army Corp of Engineers is necessary to adequately design a slope protection groin installation. For a more complete discussion on groins, designers should consult Volume II, Chapter 6, Section VI, of the Corps' Shore

Typical Groin Layout With Resultant Beach Configuration



Long Groins Without Revetment



Short Groins With Light Stone Revetment

NOTE:

"S", "L" and "θ" are determined by conditions at site.

Figure 873.4C

Protection Manual until Part VI of the Coastal Engineering Manual is published. Preliminary studies can be made by using basic information and data available from USGS quadrangle sheets, USC & GS navigation charts, hydrographic charts on currents for the Northeast Pacific Ocean and aerial photos of the area.

Factors pertinent to design include:

(1) Alignment. Factors which influence alignment are effectiveness in detaining littoral drift, and self-protection of the groin against damage by wave action.

A field of groins acts as a series of headlands, with beaches between each pair aligned in echelon, that is, extending from outer end of the downdrift groin to an intermediate point on the updrift groin, see Figure 873.4D. The offset in beach line at each groin is a function of spacing of groins, volume of littoral drift, slope of sea bed and strength of the sea, varying measurably with the season. Length and spacing must be complementary to assure continuity of beach in front of a highway embankment.

A series of parallel spurs normal to the beach extending seaward would be correct for a littoral drift alternating upcoast and downcoast in equal measure. However, if drift is predominantly in one direction the median attack by waves contributes materially to the longshore current because of oblique approach. In that case the groin should be more effective if built oblique to the same degree. Such an alignment will warrant shortening of the groin in proportion to the cosine of the obliquity, see Figure 873.4D.

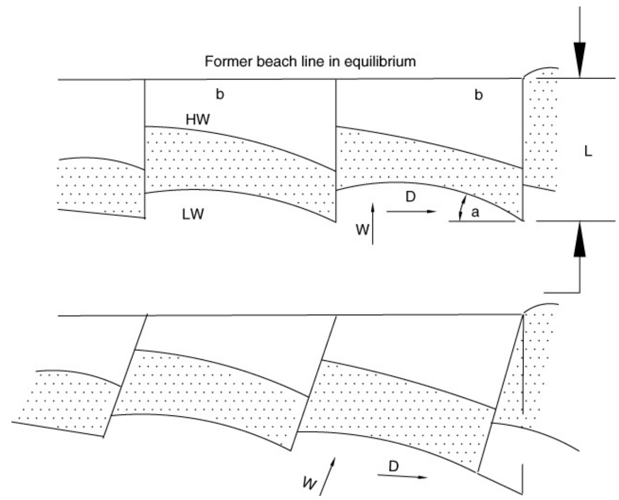
Conformity of groin to direction of approach of the median sea provides an optimum ratio of groin length to spacing, and the groin is least vulnerable to storm damage. Attack on the groin will be longitudinal during a median sea and oblique on either side in other seas.

(2) Grade. The top of groins should be parallel to the existing beach grade. Sand may pass over a low barrier. The top of the groin should be established higher than the existing beach, say 2 feet as a minimum for moderate exposure combined with an abundance of littoral drift, to 5 feet for severe exposure and deficiency of littoral drift.

The shore end should be tapered upward to prevent attack of highway embankment by rip currents, and the seaward end should be tapered downward to match the side slope of the groin in order to diffuse the direct

Figure 873.4D

Alignment of Groins to an Oblique Sea Warrants Shortening Proportional to Cosine of Obliquity



attack of the sea on the end of the groin.

(3) Length and Spacing. The length of groin should equal or exceed the sum of the offset in shoreline at each groin plus the width of the beach from low water (LW) to high water (HW) line, see Figure 873.4D. The offset is approximately the product of the groin spacing and the obliquity (in radians) of the entrapped beach. The width of beach is the product of the slope factor and the range in stage. The relation can be formulated:

$$L = ab + rh$$

Where:

L = Length of groin, feet

a = obliquity of entrapped beach in radians

b = beach width between groins, feet

r = reciprocal of beach slope

h = range in stage, feet

For example, with groins 400 feet apart, obliquity up to 20 degrees, on a beach sloping 10:1 with a tidal range of 11 feet,

L = .35 x 400 + 10 x 11 = 250 feet

The same formula would have required L = 390 feet for 800-foot spacing, reducing the aggregate length of groins but increasing the depth of water at the outer ends and the average cost per foot. For some combination of length and spacing the total cost will be a minimum, which should be sought for economical design.

If groins are too short, the attack of the sea will still reach the highway embankment with only some reduction of energy. Some sites may justify a combination of short groins with light revetment to accommodate this remaining energy.

(4) Section. The typical section of a groin is shown in Figure 873.4E. The stone may be specified as a single class, or by designating classes to be used as bed, core, face and cap stones.

Face stone may be chosen one class below the requirement for revetment by Chart A or B, Figure 873.3G. Full mass stone should be specified for bed stones, for the front face at the outer end of the groin, and for cap stones exposed to overrun. Core stones in wide groins may be smaller.

Width of groin at top should be at least 1.5 times the diameter of cap stones, or wider if necessary for operation of equipment. Side slopes should be 1.5:1 for optimum economy and ordinary stability. If this slope demands heavier stone than is available, side slope can be flattened or the cap and face stones bound together with concrete as shown in Figure 873.3F.

(e) Baffle. A baffle is a pier, vane, sill, fence, all or mound built on the bed of a stream to control, deflect, check or disturb the flow or to float on the surface to dampen wave action.

Baffles typically take the following forms of construction:

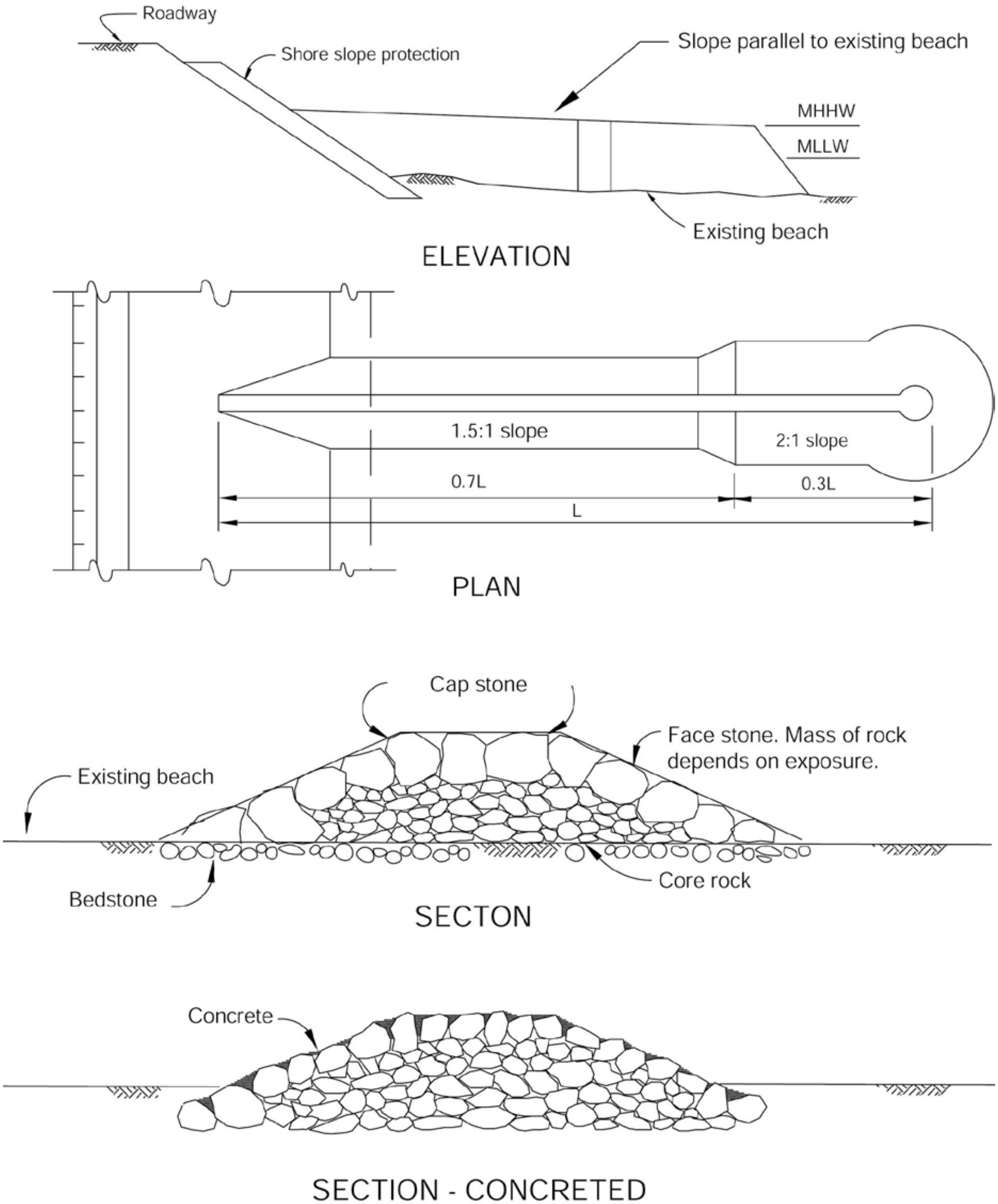
- Single or multiple lines of fence.
- Drop Structures (gabions, rock, concrete, etc.).
- Dikes of earth or rock.
- Floating boom.

These devices may vary in magnitude from a check dam on a small stream to a system of training dikes or permeable jetties for deflecting or directing flow. When using fences, palisades, or dikes as deflectors along the more mature valleys or meandering streams, the potential erosion to previously unexposed areas, threat to adjacent property, eddy currents and possibility of scour should all be assessed. When used as a collecting system to control and direct the flow to new or existing drainage facilities or to bridge openings, the alignment of the installation should be developed as a series of curves and intervening tangents guiding the stream through transitions to maintain smooth and steady flow. The surface and curvature of the training device should be governed by the natural or modified velocity.

Drop structures or check dams are an effective means of gradient control. They may be constructed of rock, gabions, concrete, timber, sacked concrete, filled fences, sheet piling or combinations of any of the above. They are most suited to locations where bed materials are relatively impervious otherwise underflow must be prevented by cutoffs. Refer to HDS No. 6, Section 6.4.11, for further discussion on the use of drop structures.

Floating booms are effective protection against the smaller wave actions common to lakes and tidal basins. Anchorage is the prime structural consideration.

Figure 873.4E
Typical Stone Dike Groin Details



NOTES:

This is not a standard design.

Dimensions and details should be modified as required.

873.5 Design Check List

The designer should anticipate the more significant problems that are likely to occur during the construction and maintenance of channel and shore protection facilities. So far as possible, the design should be adjusted to eliminate or minimize those potential problems.

The logistics of the construction activity such as access to the site, on-site storage of construction materials, time of year restrictions, environmental concerns, and sequence of construction should be carefully considered during the project design. The stream and shoreline morphology and their response to construction activities are an integral part of the planning process. Communication between the designer and those responsible for construction administration as well as maintenance are important.

Channel and shore protection facilities require periodic maintenance inspection and repair. Where practicable, provisions should be made in the facility design to provide access for inspection and maintenance.

The following check list has been prepared for both the designer and reviewer. It will help assure that all necessary information is included in the plans and specifications. It is a comprehensive list for all types of protection. Items pertinent to any particular type can be selected readily and the rest ignored.

1. Location of the planned work with respect to:
   - The highway.
   - The stream or shore.
   - Right of way.
2. Datum control of the work, and relation of that datum to gage datum on streams, and both MSL and MLLW on the shore.
3. A typical cross section indicating dimensions, slopes, arrangement and connections.
4. Quantity of materials (per foot, per protection unit, or per job).
5. Relation of the foundation treatment with respect to the existing ground.
6. Relation of the top of the proposed protection to design high water (historic, with date; or predicted, with frequency).
7. The limits of excavation and backfill as they may affect measurement and payment.
8. Construction details such as weep holes, rock slope protection fabrics, geocomposite drains and associated materials.
9. Location and details of construction joints, cut-off stubs and end returns.
10. Restrictions to the placement of reinforcement.
11. Connections and bracing for framing of timber or steel.
12. Splicing details for timber, pipe, rails and structural shapes.
13. Anchorage details, particularly size, type, location, and method of connection.
14. Size, shape, and special requirements of units such as precast concrete shapes and other manufactured items.
15. Number and arrangement of cables and details of fastening devices.
16. Size, mass per unit area, mesh spacing and fastening details for wire-fabric or geosynthetic materials.
17. On timber pile construction the number of piles per bent, number of bents, length of piling, driving requirements, cut-off elevations, and framing details.
18. On fence-type construction the number of lines or rows of fence, spacing of lines, dimensions of posts, details of bracing and anchorage ties, details of ties at end.
19. The details of gabions and the filling material.
20. The size of articulated blocks, the placement of steel, and construction details relating to fabrication.
21. The corrosion considerations that may dictate specialty concretes, coated reinforcing, or other special requirements.

Depending on the location, average maintained horizontal illumination levels of 5 lux to 22 lux should be considered. Where special security problems exist, higher illumination levels may be considered. Light standards (poles) should meet the recommended horizontal and vertical clearances. Luminaires and standards should be at a scale appropriate for a pedestrian or bicycle path. For additional guidance on lighting, consult with the District Traffic Electrical Unit .

1003.2 Class II Bikeways (Bike Lanes)

Design guidance that address the safety and mobility needs of bicyclists on Class II bikeways (bike lanes) is distributed throughout this manual where appropriate.

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

1003.3 Class III Bikeways (Bike Routes)

Class III bikeways (bike routes) are intended to provide continuity to the bikeway system. Bike routes are established along through routes not served by Class I or II bikeways, or to connect discontinuous segments of bikeway (normally bike lanes). Class III facilities are facilities shared with motor vehicles on the street, which are established by placing bike route signs along roadways. Additional enhancement of Class III facilities can be provided by adding shared roadway markings along the route. For application and placement of signs and pavement markings, see the California MUTCD Section 9C.

Minimum widths for Class III bikeways are represented, in the minimum standards for highway lanes and shoulder.

Since bicyclists are permitted on all highways (except prohibited freeways), the decision to designate the route as a bikeway should be based on the advisability of encouraging bicycle travel on the route and other factors listed below.

(1) *On-street Bike Route Criteria*. To be of benefit to bicyclists, bike routes should offer a higher degree of service than alternative streets. Routes should be signed only if some of the following apply:

   (a) They provide for through and direct travel in bicycle-demand corridors.

   (b) Connect discontinuous segments of bike lanes.

   (c) They provide traffic actuated signals for bicycles and appropriate assignment of right of way at intersections to give greater priority to bicyclists, as compared with alternative streets.

   (d) Street parking has been removed or restricted in areas of critical width to provide improved safety.

   (e) Surface imperfections or irregularities have been corrected (e.g., utility covers adjusted to grade, potholes filled, etc.).

   (f) Maintenance of the route will be at a higher standard than that of other comparable streets (e.g., more frequent street sweeping).

(2) *Sidewalk as Bikeway*. Sidewalks are not to be designated for bicycle travel. Wide sidewalks that do not meet design standards for bicycle paths or bicycle routes also may not meet the safety and mobility needs of bicyclists. Wide sidewalks can encourage higher speed bicycle use and can increase the potential for conflicts with turning traffic at intersections as well as with pedestrians and fixed objects.

   In residential areas, sidewalk riding by young children too inexperienced to ride in the street is common. It is inappropriate to sign these facilities as bikeways because it may lead bicyclists to think it is designed to meet their safety and mobility needs. Bicyclists should not be encouraged (through signing) to ride their bicycles on facilities that are not designed to accommodate bicycle travel.

(3) *Shared Transit and Bikeways.* Transit lanes and bicycles are generally not compatible, and present risks to bicyclists. Therefore sharing exclusive use transit lanes for buses with bicycles is discouraged.

   Bus and bicycle lane sharing should be considered only under special circumstances to provide bikeway continuity, such as:

(a) If bus operating speed is 25 miles per hour or below.

(b) If the grade of the facility is 5 percent or less.

1003.4 Trails

Trails are generally, unpaved multipurpose facilities suitable for recreational use by hikers, pedestrians, equestrians, and off-road bicyclists. While many Class I facilities are named as trails (e.g. Iron Horse Regional Trail, San Gabriel River Trail), trails as defined here do not meet Class I bikeways standards and should not be signed as bicycle paths. Where equestrians are expected, a separate equestrian trail should be provided. See DIB 82 for trail requirements for ADA. See Index 208.7 for equestrian undercrossing guidance.

- Pavement requirements for bicycle travel are not suitable for horses. Horses require softer surfaces to avoid leg injuries.
- Bicyclists may not be aware of the need to go slow or of the separation need when approaching or passing a horse. Horses reacting to perceived danger from predators may behave unpredictably; thus, if a bicyclist appears suddenly within their visual field, especially from behind they may bolt. To help horses not be surprised by a bicyclist, good visibility should be provided at all points on equestrian paths.
- When a corridor includes equestrian paths and Class I bikeways, the widest possible lateral separation should be provided between the two. A physical obstacle, such as an open rail fence, adjacent to the equestrian trail may be beneficial to induce horses to shy away from the bikeway, as long as the obstacle does not block visibility between the equestrian trail and bicycle path.

See FHWA-EP-01-027, Designing Sidewalks and Trails for Access and DIB 82 for additional design guidance.

1003.5 Miscellaneous Criteria

The following are miscellaneous bicycle treatment criteria. Specific application to Class I, and III bikeways are noted. Criteria that are not noted as applying only to bikeways apply to any highway, roadways and shoulders, except freeways where bicycles are prohibited), without regard to whether or not bikeways are established.

Bicycle Paths on Bridges – See Topic 208.

(1) *Pavement Surface Quality*. The surface to be used by bicyclists should be smooth, free of potholes, and with uniform pavement edges.

(2) *Drainage Grates, Manhole Covers, and Driveways*. Drainage inlet grates, manhole covers, etc., should be located out of the travel path of bicyclists whenever possible. When such items are in an area that may be used for bicycle travel, they shall be designed and installed in a manner that meets bicycle surface requirements. See Standard Plans. They shall be maintained flush with the surface when resurfacing.

If grate inlets are to be located in roadway or shoulder areas (except freeways where bicycles are prohibited) the inlet design guidance of Index 837.2(2) applies.

Future driveway construction should avoid construction of a vertical lip from the driveway to the gutter, as the lip may create a problem for bicyclists when entering from the edge of the roadway at a flat angle. If a lip is deemed necessary, the height should be limited to ½ inch.

(3) *At-grade Railroad Crossings and Cattle Guards*. Whenever it is necessary for a Class I bikeway, highway or roadway to cross railroad tracks, special care must be taken to ensure that the safety of users is protected. The crossing must be at least as wide as the traveled way of the facility. Wherever possible, the crossing should be straight and at right angles to the rails. For bikeways or highways that cross tracks and where a skew is unavoidable, the shoulder or bikeway should be widened, to permit bicyclists to cross at right angles (see Figure 1003.5). If this is not possible, special construction and materials should be considered to keep the flangeway depth and width to a minimum.

Pavement should be maintained so ridge buildup does not occur next to the rails. In

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