# Local Drainage Manual

## County of Orange
## Department of Public Works

James Treadaway, P.E. and S.E.
Director of OC Public Works

Approved and Adopted: May 25, 2021
by
Orange County Board of Supervisors

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# County of Orange

**OC LOCAL DRAINAGE MANUAL, 2ND EDITION**

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# Abbreviations and Acronyms

## Acronyms

| | |
|---|---|
| **ACPA** | **American Concrete Pipe Associations** |
| **APWA** | **American Public Works Association** |
| **AASHTO** | **American Association of State Highway and Transportation Officials** |
| **Caltrans** | **California Department of Transportation** |
| **CCC** | **California Coastal Commission** |
| **CDFW** | **California Department of Fish and Wildlife** |
| **CWA** | **Clean Water Act** |
| **DAMP** | **Drainage Area Management Plan** |
| **DSOD** | **Division of Safety of Dams** |
| **DWQ** | **Division of Water Quality** |
| **FEMA** | **Federal Emergency Management Agency** |
| **FIRM** | **Flood Insurance Rate Map** |
| **LOMR** | **Letter of Map Revision** |
| **MS4** | **Municipal Separate Storm Sewer System** |
| **NFIP** | **National Flood Insurance Program** |
| **NPDES** | **National Pollutant Discharge Elimination System** |
| **OCPW** | **Orange County Public Works** |
| **OCFCD** | **Orange County Flood Control District** |
| **PMR** | **Physical Map Revision** |
| **RWQCB** | **Regional Water Quality Control Board** |
| **SSPWC** | **Standard Specifications for Public Works Construction** |
| **SWRCB** | **State Water Resources Control Board** |
| **SWPPP** | **Storm Water Pollution Prevention Plan** |
| **USACE** | **United States Army Corps of Engineers [ACOE]** |
| **USBR** | **United States Bureau of Reclamation** |
| **USEPA** | **United States Environmental Protection Agency** |

**Pipe Material Acronyms are in Table 4-3: Storm Drain Designations.**

# Design Abbreviations

| | |
|---|---|
| **a** | **depth of depression in feet** |
| **A** | **area in square feet** |
| **A2-6** | **6 inch high concrete curb** |
| **A2-8** | **8 inch high concrete curb** |
| **Crown** | **highest point in street** |
| **cfs** | **cubic feet per second** |
| **CSP** | **Corrugated Steel Pipe** |
| **D** | **diameter of pipe in inches** |
| $D_1$ | **depth from invert to HGL** |
| $H_{minor}$ | **minor head loss** |
| **EGL** | **Energy Grade Line** |
| **HGL** | **Hydraulic Grade Line** |
| **L** | **length in feet** |
| **D/S** | **down stream** |
| $L_p$ | **length of practical intersection of flow** |
| **fps** | **feet per second** |
| **LRFD** | **Load and Resistance Factor Design** |
| **mph** | **miles per hour** |
| **Q** | **discharge in cubic feet per second** |
| $Q_p$ | **partial Q interception** |
| **R/W** | **right of way** |
| **S** | **street longitudinal slope** |
| $S_x$ | **street transverse slope** |
| **100-year protection** | **a given elevation for 100-year discharge** |
| **R** | **hydraulic radius** |
| **n** | **Manning's roughness coefficient** |
| $Y \times V$ | **depth times velocity (safe depth for pedestrians)** |
| **Y** | **depth in gutter in feet** |
| **y** | **depth in water in feet** |

# Chapter 1 Introduction

## 1.1 Purpose and Scope

The Orange County Local Drainage Manual provides design criteria policies and procedures for engineers to use as guidelines and is intended to exercise sound judgment in the design of drainage facilities in Orange County. The manual is intended neither to be used as, nor to establish legal standards for these functions. State and Federal standards shall supersede any standard provided by this manual as applicable (Example: Americans with Disabilities Act of 1990).

The primary purpose of the Orange County Local Drainage Manual is to provide a minimum standard for the design of local drainage systems (tributary areas up to 640 acres) built for dedication to the County of Orange or private drainage facilities within unincorporated Orange County. This manual provides design guidance to local jurisdictions, design engineers, and environmental professionals in the selection and design of drainage facilities. This manual is not intended to supersede any information contained within the Orange County Drainage Area Management Plan (DAMP).

This manual is intended to provide guidance in the design of new and major reconstruction projects. The fact that new minimum design values are presented does not imply that existing facilities are in any way inadequate. The values contained herein were updated to conform to new trends in the industry.

The County assumes no responsibility for design of facilities adhering to the standards contained hereon. Review and approval do not absolve the owner, developer, design engineer, of responsibilities for design. The design engineer has the responsibility to design drainage facilities that meet standards of practice for the industry and promote public safety. Compliance with the regulatory elements and meeting the policies and minimum design standards do not constitute a guarantee that properties will be free from flooding or flood damage. The County and their officials or employees assume no responsibility for information, data, or conclusions prepared by private engineers or environmental professionals and make no warranty expressed or implied in their review/approval of local drainage projects.

The manual discusses the following topics:

- Submittal Requirements
- Street Drainage and Inlets
- Storm Drains
- Culverts
- Open Channels
- Detention Basins
- Energy Dissipators
- Debris Facilities
- Structures
- Floodplains

## 1.2 General Protection Requirements

The goal of this manual is to provide 100-year flood protection for all habitable structures pursuant to the Public Services and Facilities Element of the County General Plan. Specific protection and freeboard criteria for each facility are located in the corresponding facility chapter. Additional information pertaining to freeboard is located in Appendix F.

## 1.3 Policies

For all design, staff engineers shall exercise “Standard of Ordinary Care” as would be expected of members of the profession in carrying out assigned projects. The term “Standard of Ordinary Care” is not defined and is a matter to be considered with each specific design. However, it is OCPW policy that design may be considered to have a “Standard of Ordinary Care” if the design follows the Standard Plans, Local Drainage Manual, other approved references, or has approved deviations from these documents, which are supported by sound engineering or safety considerations.

In cases where strict adherences to the standards of design would be impractical or unreasonable, deviations may be approved provided they are in accordance with good engineering practice and the public health and safety, and conform to a plan that will, under the circumstances in such case, be practical and reasonable. Any deviations from the standards of design shall be identified by a note on the plans and approval of these plans shall constitute approval of these deviations. All deviations shall include supporting documentation and justification and be placed in the appropriate County design file. Special circumstances, which may be cited to justify deviation from the standards, include but are not limited to the character of the community, alternative means of storm flow protection, environmental considerations, physical constraints, existing nearby uses, and economic considerations.

If there is a conflict between design policies, then the policy highest in precedence shall prevail. The precedence of design documents shall be:

1. Orange County Codified Ordinances
2. OC-LDM
3. OCFCD Design Manual
4. Orange County Highway Design Manual
5. Orange County Standard Plans
6. Other Orange County Design Manuals

This manual is not a textbook or a substitute for engineering knowledge, experience or judgment. No attempt is made to detail basic engineering techniques.

<u>Application of Design Standards</u>

Orange County’s policy is that local drainage facilities adhere to the criteria covered in this manual. Other agencies and design engineers must use this manual in planning new facilities and in their review of proposed work where the County has discretionary approval of the project.

<u>Exceptions to Design Standards</u>

The design standards and procedures included in this manual have been compiled and reviewed so that they meet the County’s design standards and are applicable to most situations. Therefore, exceptions to the Design Standards should be rare and must be approved by the Chief Engineer.

<u>Update of Design Standards</u>

The criteria in this manual will be reviewed, revised, and updated as necessary to reflect current drainage design standards.

## 1.4 Drainage Law

California Drainage Law (Government Code Section 65300 – 65303.4) is essentially Case law, therefore, it is complex, but the courts have established the following general principles, which apply in general to development projects:

• The upstream property owner shall not concentrate water where it was not concentrated before without making proper provision for its disposal without damage to the downstream property owner.

• The upstream property owner may not further increase drainage runoff by diversion of water that previously drained to another area. Reasonableness is often based on prevailing standards of practice in the community or region.

• The downstream property owner is obligated to accept and make provisions for those waters that are the natural flow from the land above.

• No property owner shall block or permit to be blocked any drainage channel, ditch, or pipe. No property owner shall divert drainage water without properly providing for its disposal.

The historical case law concerning drainage is too extensive to list within this manual. California drainage law is based on the following "good neighbor" policies.

- Landowners have the right to discharge water in a reasonable manner.
- Downstream landowners must accept upstream landowners' naturally flowing surface water.
- Landowners must take reasonable care to avoid damage to adjacent properties due to runoff.

## 1.5 Authority and Jurisdiction of the County of Orange and the Orange County Flood Control District

The Orange County Flood Control District (OCFCD), established May 23, 1927 under authorization of the Orange County Flood Control Act, Chapter 723 of the State of California Statutes of 1927, was created to

- Provide control of flood and storm waters of the district (the boundary of the County of Orange) and of streams flowing into the district (such as the Santa Ana River or San Juan Creek);
- Mitigate the effects of tides and waves; and
- Protect the harbors, waterways, public highways and property in the district from such waters.

OCFCD is administered by the Orange County Public Works (OCPW) and is governed by the Orange County Board of Supervisors. OCFCD is a political entity that has no employees but owns land and assesses an annual benefit on all taxable real property in Orange County. Because OCFCD has no employees, the District and its property are administered, maintained, and operated by OCPW staff who are in turn employed by the County of Orange.

OCPW staff regulates development within the County jurisdiction and oversees the design and construction of local drainage facilities. OCPW regulates local drainage based on the following references:

- <u>State of California Subdivision Map Act</u>
  - Division 2 Chapter 4 Article 1 §66474.7
  - Division 2 Chapter 4 Article 5 §66483
  - Division 2 Chapter 4 Article 6 §66488
- <u>Orange County Code of Regulatory Ordinances</u>
  - County Building Code (Section 7-1-12)
  - FP "Floodplain" District Regulations (Section 7-9-42 and 7-9-292)
  - Grading Code (Article 8)

- Drainage Fees (Sections 7-9-63.2 and 7-9-315)

### 1.5.1 Drainage System Classification

Understanding the facility classifications used by the County will be very helpful to the design engineer.

There are four general classifications of drainage facilities/ownership within Orange County. Refer to Chapter 2: Submittal Requirements for each classification's individual filing guidelines. The four general classifications are as follows:

#### 1.5.1.1 Regional/OCFCD Drainage Facilities

Regional facilities are usually owned, maintained and operated by OCFCD. Regional facilities are classified by the number of acres in its watershed. The minimum size of a tributary drainage area for a regional facility is 1000 acres.

Defining a facility as regional should also be determined on a logical basis. At the 1000-acre cutoff point, the watercourse should be examined upstream and downstream for the locations at which ownership changes. Such locations may include confluences, transitions from a storm drain to an open channel, changes from street R/W to channel R/W, locations of access, city boundaries, or other factors.

The designer is directed to the Orange County Flood Control Design Manual (OCFCD Design Manual) for regional facility design criteria.

OCFCD Facilities are titled by the drainage area. For example, Santa Ana-Delhi Channel is located in Watershed "F" and is facility number 1. Therefore, it would be filed under F01. The OCFCD drawing number would be F01-XXX-XX. Other sequence numbers pertain to the plans' filing system and will be assigned by OCPW.

#### 1.5.1.2 Sub-Regional Drainage Facilities

Sub-regional facilities are usually owned, maintained and operated by OCFCD. Sub-regional facilities are classified by the number of acres in its watershed. The minimum size for a sub-regional facility is 640 acres and the maximum size is 1000 acres.

Defining a facility as sub-regional should also be determined on a logical basis. At the 640-acre and 1000-acre cutoff points, the watercourse should be examined upstream and downstream for the locations at which ownership changes. Such locations may include confluences, transitions from a storm drain to an open channel, changes from street R/W to channel R/W, locations of access, city boundaries, or other factors.

The designer is directed to the OCFCD Design Manual for design criteria.

OCFCD Sub-regional Facilities are titled by the drainage area and receiving drainage facility. For example, Santa Ana-Delhi Channel (a regional facility) is in Watershed "F" and is facility number 1. It has the designation of facility number F01. The Airport Storm Drain is a sub-regional facility (draining 640 to 1000 acres) that drains into the Santa Ana-Delhi Channel. Sub-regional facilities are indicated with the letter "S" and a number (e.g. S01, S02, etc.). The Airport Storm Drain therefore receives the designation of facility number F01S01 and would be filed under F01S01-XXX-XX. The other sequence numbers pertain to the plans' filing system and would be assigned by OCPW.

#### 1.5.1.3 Local Drainage Facilities

Local drainage facilities are publicly owned drainage facilities where they establish a public benefit, meet local agency standards, meet flood protection goals and are dedicated to and accepted by the local agency. Local drainage facilities are classified as having a watershed less than 640 acres, and thus are generally smaller than sub-regional facilities.

Local facilities can be owned by a city or by the County. If County owned, the ownership will pass to a city upon incorporation or annexation. Therefore, before plan preparation begins, the design engineer should discuss ownership issues with the County, and separate tract improvement plans should be filed with the County and will be given to the City upon incorporation.

Local drainage facilities are titled by the drainage area and pipe location. For example, a local facility (draining less than 640 acres) would be labeled P01, P02, etc. If the local facility connects to the Santa Ana-Delhi channel (a regional facility in Watershed “F”), then the facility name becomes F01P01. An example drawing number would be F01P01-XXX-XX. Other sequence numbers pertain to the plans’ filing system and will be assigned by OC Development Services.

#### 1.5.1.4 Private Drainage Facilities

Private facilities are those that do not provide enough public benefit to be considered for public ownership. Private drainage facilities are generally smaller than local drainage facilities, and are generally owned and maintained by a landowner, homeowner association, or other non-public organization.

The facility would be included in the Development Plans. No facility number would be assigned.

### 1.5.2 MS4 NPDES Permits

The Municipal Separate Storm Sewer Systems (MS4) National Pollutant Discharge Elimination System (NPDES) Permits require the County to develop programs to control pollutants in storm water discharge from the MS4. These permits are issued to the County and OCFCD by the State of California pursuant to the Federal Clean Water Act and California's Porter Cologne Water Quality Act. The County is subject to two different NPDES Permits that are renewed approximately every five years: 1) Santa Ana Regional Water Quality Control Board Order No. R8-2009-0030 as amended by Order No. R8-2010-0062 and 2) San Diego Regional Water Quality Control Board Order No. R9-2013-0001 as amended by Order Nos. R9-2015-001 and R9-2015-0100. Guidance for complying with these permits is included in the current version of the Orange County Drainage Area Management Plan (DAMP). This manual is intended to complement the DAMP. For the most up to date definition for an MS4 please visit the California Water Boards website: https://www.waterboards.ca.gov/water_issues/programs/stormwater/municipal.html.

## 1.6 Other Federal and State Regulations

### 1.6.1 Federal Emergency Management Agency

The County is a participant in the National Flood Insurance Program (NFIP) and the Community Rating System (CRS), which is administered by the Federal Emergency Management Agency (FEMA). The County’s participation in the CRS requires the County to adopt minimum floodplain development standards, which are reflected in this manual and the OCFCD Design Manual, in exchange for reduced flood insurance rates for properties within the County’s jurisdictional limits.

### 1.6.2 Clean Water Act

The Clean Water Act of 1972 (CWA), as amended by the Water Quality Act of 1987, is the major federal legislation governing water quality, which was enacted "to restore and maintain the chemical, physical, and biological integrity of the nation's waters" ( CWA § 101). The aforementioned MS4 NPDES permits are required pursuant to the CWA.

### 1.6.3 Porter-Cologne Water Quality Control Act

The Porter-Cologne Act established the State Water Resources Control Board (SWRCB or State Board) and the nine Regional Water Quality Control Boards (RWQCBs or Regional Boards), and authorized the State Board to formulate, adopt, and revise state water policy, which may include water quality objectives, principles, and guidelines. In addition, it authorizes the State Board to adopt water quality control plans on its own initiative, which supersede Regional Water Quality Control Plans to the extent of any conflict. Article 3 of the Porter-Cologne Act directs Regional Boards to adopt, review, and revise Water Quality Control Plans (Basin Plans) and provides specific guidance on factors that must be considered when adopting water quality objectives and implementation measures. It also allows Regional Boards to prohibit discharges in Basin Plans or in waste discharge requirements.

### 1.6.4 Construction General Permit

The Statewide NPDES Construction General Permit for storm water discharges associated with construction and land disturbance activities, Order 2009-0009-DWQ as amended by 2010-0014-DWQ and 2012-0006-DWQ, requires that any construction project disturbing more than one acre of land obtain coverage for any size parcel that is part of a larger common plan of development or for any project site that the RWQCB requires coverage.

### 1.6.5 Construction Dewatering Permits

Dewatering activities will require a permit. Some of these possibilities include the discharge of stranded construction storm water, discharge of groundwater to land, and discharge of groundwater to a surface water. The specific nature of each situation must be independently evaluated. The design engineer should begin their review on the appropriate Regional Water Quality Control Board webpage and then seek confirmation of the appropriate approach with Regional Board staff.

### 1.6.6 Industrial General Permit

The Statewide NPDES Industrial General Permit for storm water discharges associated with industrial activities, Order 2014-0057-DWQ, is an NPDES permit that regulates discharges associated with industrial facilities. The Industrial General Permit requires the implementation of management measures that will achieve the performance standards. The Industrial General Permit also requires the development of a Storm Water Pollution Prevention Plan (SWPPP) and a monitoring plan.

### 1.6.7 California Division of Safety of Dams

The Department of Water Resources Division of Safety of Dams (DSOD) administers the state's dam safety program to protect people against the loss of life and property from dam failure. The OCPW Standard Plan 1327 Exhibit 1 (Figure 7-1 in this manual) shows the jurisdictional dam size for local grading permits, local (city or county) jurisdictional size and state (DSOD) jurisdictional size.

### 1.6.8 Regulatory Permits

#### 1.6.8.1 U.S. Army Corps of Engineers

The U.S. Army Corps of Engineers (USACE) regulates discharges of dredged or fill materials into waters of the U.S. and wetlands pursuant to Section 404 of the federal Clean Water Act. An approval (i.e., Nationwide Permit or Individual Permit) may be required from the USACE before commencement of any construction activities (i.e., dredge or fill) within USACE delineated jurisdictional areas.

The USACE also requires a Section 408 permit for approval of alteration of existing U.S. Army Corps of Engineers public works projects. Public works projects include U.S. Army Corps of Engineers dams and local flood protection works constructed by the United States for which Non-Federal Sponsors (State, City, or other agencies such as OCFCD) have the responsibilities for operation and maintenance.

#### 1.6.8.2 Regional Water Quality Control Boards

The respective Regional Water Quality Control Board (RWQCB) regulates discharges to surface waters under the federal Clean Water Act and the California Porter-Cologne Water Quality Control Act. For a Corps 404 permit to be approved a Section 401 Water Quality Certification from the RWQCB will be required.

#### 1.6.8.3 California Department of Fish and Wildlife

The California Department of Fish and Wildlife (CDFW) regulates alteration to streambeds and associated vegetation under the Fish and Game Code. The CDFW must be notified before activities that alter jurisdictional areas. A Streambed Alteration Agreement from the CDFW will be required before commencement of any construction activities within CDFW delineated jurisdictional areas.

#### 1.6.8.4 California Coastal Commission

For projects in or affecting the coastal zone, the California Coastal Commission (CCC) regulates development activities pursuant to the Coastal Act. A Coastal Development Permit from the CCC must be obtained before impacts occurring within CCC jurisdictional areas.

## 1.7 Document Organization

The Orange County Local Drainage Manual is divided into 11 chapters that address the design of local drainage (i.e. facilities serving tributary areas less than 640 acres). Facility drainage areas greater than 640 acres shall be designed in accordance with the OCFCD Design Manual.

This manual includes the scope, policies, laws, regulations, and methods of using the manual. Chapter 2 discusses the submittal requirements for local drainage projects. Of the 11 chapters, seven are devoted to facility hydraulic design requirements (Chapters 3-9). Each of the facility chapters are organized so that general design criteria are contained in the second subsection of the chapter and specific facility hydraulic design considerations including example calculations are provided. Each facility chapter also concludes with a list of acceptable software and references. Chapter 10 is dedicated to structural design requirements for the local drainage structures included in Chapters 3 through 9. Chapter 11 explains the requirements and procedures for construction in and around FEMA floodplains.

## 1.8 List of County Manuals

Although this 2$^{nd}$ Edition attempts to be comprehensive for development of local drainage systems, the following County resources may be of use:

- Current edition of the Orange County Public Works Department Standard Plans
- Current edition of the Orange County Hydrology Manual
- Current edition of the Orange County Drainage Area Management Plan
- Current edition of the Orange County Flood Control District Design Manual

All references to County resources within this document refer to the current edition. Other resources referenced throughout the document may be consulted to obtain more detailed information on specific topics, such as FHWA HEC-14 for energy dissipation.

## 1.9 Use of Standard Drawings

This manual incorporates by reference the Orange County Public Works Department Standard Plans. This manual and the standard plans are intended to be used to design and construct local drainage systems in Orange County. When there is no OC Public Works Standard Plan available for the design situation, possible additional resources for standard plans include Standard Plans for Public Works Construction (SPPWC), California Department of Transportation (Caltrans) and Los Angeles County. The only standard plans guaranteed to be accepted by the County are the Orange County Public Works Department Standard Plans. Standard plans from other resources must be reviewed and approved by OCPW.

## 1.10 References

California Department of Fish and Wildlife. (n.d.). *Lake and Streambed Alteration Program*. https://wildlife.ca.gov/Conservation/LSA

California Coastal Commission. (2019). *California Coastal Act of 1976*. https://www.coastal.ca.gov/coastact.pdf

Federal Emergency Management Agency. (2020, April). *National Flood Insurance Program Community Rating System*. https://www.fema.gov/national-flood-insurance-program-community-rating-system

Orange County Public Works. (n.d.) *Documents and Maps*. https://ocip.ocpublicworks.com/service-areas/oc-infrastructure-programs/documents-maps

Orange County Public Works. (2018). *Standard Plans (2018 Edition)*. https://www.ocpublicworks.com/about/oc_public_works_standard_plans

State Water Resources Control Board. *Municipal Stormwater Program*. https://www.waterboards.ca.gov/water_issues/programs/stormwater/municipal.html

State Water Resources Control Board. (2012). *National Pollutant Discharge Elimination System (NPDES) General Permit for Storm Water Discharges Associated with Construction and Land Disturbance Activities*. http://www.waterboards.ca.gov/water_issues/programs/stormwater/constpermits.shtml

State Water Resources Control Board. (2014). *National Pollutant Discharge Elimination System (NPDES) General Permit for Storm Water Discharges Associated with Industrial Activities*. http://www.waterboards.ca.gov/water_issues/programs/stormwater/industrial.shtml

State Water Resources Control Board. (Amended 2013). *Porter-Cologne Water Quality Control Act*. State Water Resources Control Board, California Water Code §§ 13000 et seq. http://www.waterboards.ca.gov/laws_regulations/docs/portercologne.pdf

State Water Resources Control Board. (2018, April). *State and Regional Water Boards*. https://www.waterboards.ca.gov/waterboards_map.html

Thompson, P. L., & Kilgore, R. T. (2006). *Hydraulic Design of Energy Dissipators for Culverts and Channels: Hydraulic Engineering Circular Number 14 (HEC 14), Third Edition*. Federal Highway Administration. FHWA-NHI-06-086, July.

U.S. Army Corps of Engineers. (n.d.). *Application and Permit Process*. U.S. Army Corps of Engineers, Los Angeles District. https://www.spl.usace.army.mil/Missions/Regulatory/Permit-Process/

U.S. Congress, H.R. 1--92nd. 1972. *Clean Water Act of 1972 -- 86 Stat. 816*. 33 U.S.C. 1251 et seq. Code of Federal Regulations Title 40, Volume 13, Section 122: As Amended Through P.L. 107–303. http://www.epw.senate.gov/water.pdf

Water Quality Act of 1987, Publ. L. No. 1004. (1987). https://www.congress.gov/bill/100th-congress/house-bill/1

# Chapter 2 Submittal Requirements

## 2.1 General

This chapter addresses requirements of the Agency for plan check submittals. Unless the facility is being built as a condition of approval of a subdivision or other development within the limits of the subdivision and in unincorporated areas, an understanding between all parties involved, will be necessary to establish Agency requirements for design, plan check and maintenance. These items should be resolved before commencing design. When a Development condition or Agreement does not provide for County or OCFCD acceptance of the completed facility for maintenance, an Agreement is required to define both project and acceptance conditions. Developers and Design Engineers are encouraged to meet with OCPW staff to discuss these matters before commencing design.

When County or OCFCD acceptance for maintenance is intended, adequate information in the form of maps, calculations, drawings, inspection, construction and maintenance of the facility will be required. Specific submittal requirements for improvement plan checking are given later in this chapter. However, the following paragraphs describe these requirements in general terms.

### 2.1.1 Plan Notes

See Appendix B for Storm Drain Plan notes.

### 2.1.2 Hydrology, Hydraulic and Structural Calculations

Submittals shall be prepared using approved methodologies per this and other Orange County Manuals, presentation of computer outputs and/or hand calculations, and mapping that can be easily followed by the plan checker. Studies and reports shall provide for convenient filing (i.e., tract, street name, facility) that can be easily recalled in the future.

### 2.1.3 Drawings and Drafting Standards

Construction drawings shall be prepared on 22” x 34” or 24” x 36” Mylar and drawn to an appropriate scale using the OCPW Drafting Manual Standards. An electronic copy in pdf or tiff format shall also be provided for review and archiving purposes. In the future, digital format submittals may replace Mylar plan sets. Please check with the Agency to get the most current submittal standards. See Section 2.7 for further discussion on plans.

### 2.1.4 Funding Sources

Channels and storm drains are financed by private developers, County funds, Master Plan of Drainage funds or Orange County Flood Control District (OCFCD) funds. The maintenance of these facilities may be by private entities, local agencies, County or OCFCD. The entity to be responsible for maintenance of the proposed facilities shall be determined before OCPW approval of Plans and the responsible party shall be designated on the title sheet.

The County of Orange shall be shown as the maintenance entity in unincorporated areas of Orange County with public-benefiting local facilities. Where future incorporations are likely to occur, maintenance by others is transferable by ordinance or by agreement. Agreement number shall be shown on the plans. The original drawings shall provide for the separation of the appropriate plans (storm drain, streets, OCFCD) when maintenance/ownership is transferred, which may require additional title sheets. Upon annexation or incorporation, the original drawings or files for local facilities are sent to the City for ownership.

## 2.2 Existing Reference Files

Engineering drawings, maps and calculations that are submitted for drainage projects are filed in various locations within the OCPW. Clear cross-referencing of all submittals must therefore be shown to help future retrieval. Refer to Chapter 1 for classification of drainage systems. Based on the classification of the drainage system, the design engineer shall contact the corresponding parties to establish the ultimate responsibility for the facility upon its completion.

### 2.2.1 Regional and Sub-Regional OCFCD Files

Hydrology, hydraulics, and structural calculations for privately funded OCFCD projects are filed with OCPW, OC Development Services by Permit Number and/or Facility number. Hydrology calculations and mapping for publicly funded Agency projects are kept in the Flood Program Hydrology Section. Construction plans are maintained by OCPW by titled facility after completion of construction. Digital copies are archived by OCPW on the On-Base system. The designer shall contact the County to acquire the digital file. Master Planned Drainage Facility Files are retained with OC Development Services. The original drawings and files are transferred to the City after incorporation.

### 2.2.2 Local Facility Files

These files are generally filed with the Tract Improvement Plans and are referenced by the tract number. Hydrology files are retained in the OC Development Services and improvement plans are retained with OCPW. Files and plans are transferred to the City after incorporation.

### 2.2.3 Topographic, Hydrological Mapping

Maps shall be legible and at least 1” = 400’ scale.

### 2.2.4 Private Drainage Facility

Files may be included in the subdivision grading plans or subdivision improvement plans. Plans that are prepared as part of the grading process are generally filed with OCPW by street name adjacent to the tract or may be filed in the grading permit file. Current submittals are processed through the OC Development Services and saved in the permit system. Plans prepared with the tract’s street improvements will have drainage calculations filed with OC Development Services while the improvement plans are filed with OCPW.

## 2.3 Approval of Developer’s Regional Flood Control Improvements

When a developer’s project involves improvement to a watercourse of a regional nature, such improvements are outside the scope of this manual. The developer shall coordinate such efforts with OCFCD.

## 2.4 Calculations

All engineering calculations shall be prepared using this manual and any addendums or revisions. Regional facilities design shall be referenced to the latest revision of the OCFCD Design Manual and the Orange County Hydrology Manual and subsequent addendums. The design engineer’s signature with license stamp shall be included on calculation title sheet.

### 2.4.1 Hydrology Study/Calculations

A hydrology study based on the Orange County Hydrology Manual and its subsequent addendums must be submitted and approved by either the OCPW Flood Program Support Division for areas greater than

640 acres or the OCPW OC Development Services for areas less than 640 acres. All subdivision/ developer proposed projects shall be coordinated through the OC Development Services.

### 2.4.2 Hydraulic Calculations

A water surface profile based on the ultimate channel water surface shall be calculated and shown on the plans. If flow of a tributary exceeds 10% of the of the main channel flow upstream of the confluence, then the OCFCD Design Manual or OC Development Services shall be consulted.

### 2.4.3 Structural Calculations

Calculations for structures shall be submitted in a form that will allow for convenient plan checking and filing for future reference and recall. The signed and sealed report shall include cover sheet, detailed design criteria, soil data, and calculations.

## 2.5 Soils Reports

A soil report specifically addressing requirements of facility design and construction shall be submitted with the structural calculations and plans.

## 2.6 Cost Estimate

A cost estimate for construction of the facility shall be submitted with the first plan check for use in computing plan check and construction inspection fees. Cost estimates are only used for bond purposes; however, quantities and unit prices must be provided.

## 2.7 Plans

County and OCFCD funded projects are traditionally on 22” x 34” or 24” x 36” plans. The 22” x 34” plans allow a 50% reduction to 11” x 17” for reproduction. Consideration of other sizes and drafting requires the following:

- Agency approval
- Extra margin will be required on smaller plans.
- Paste-on other than Company Logos will not be accepted on original drawings.
- Title Sheet shall have Design Engineer’s signature and stamp.

## 2.8 Drafting Standards

All drafting shall conform to this manual, the OCPW Drafting Manual and CAD template updates thereof.

## 2.9 First Plan Check Submittal Form

See Appendix B for electronic references:

1) Storm Drain Plan Review Check List

2) Street Plan Check List

3) Improvement Submittal Check list

4) Hydrology and Hydraulics Check list

5) Improvement Plans

6) Storm Drain General Notes

## 2.10 Exhibits

1. A typical OCFCD title sheet (Figure 2-1) shall be used for all regional and sub-regional facilities projects maintained by the Agency on behalf of OCFCD. A blank title sheet sample is available from OCPW for duplication.
2. A typical Local Drainage Facility title sheet (Figure 2-2) shall be used for all local drainage projects maintained by the Agency on behalf of the County of Orange. A blank title sheet sample is available from OCPW for duplication.
3. A typical Plan and Profile Sheet (Figure 2-3) has been included to show placement and format of details and notes for a typical storm drain project.
4. Facilities between private and Regional (up to 640 acres) to be maintained by the County and later by the City should use Figure 2-1 with note “to be maintained by the County of Orange”.

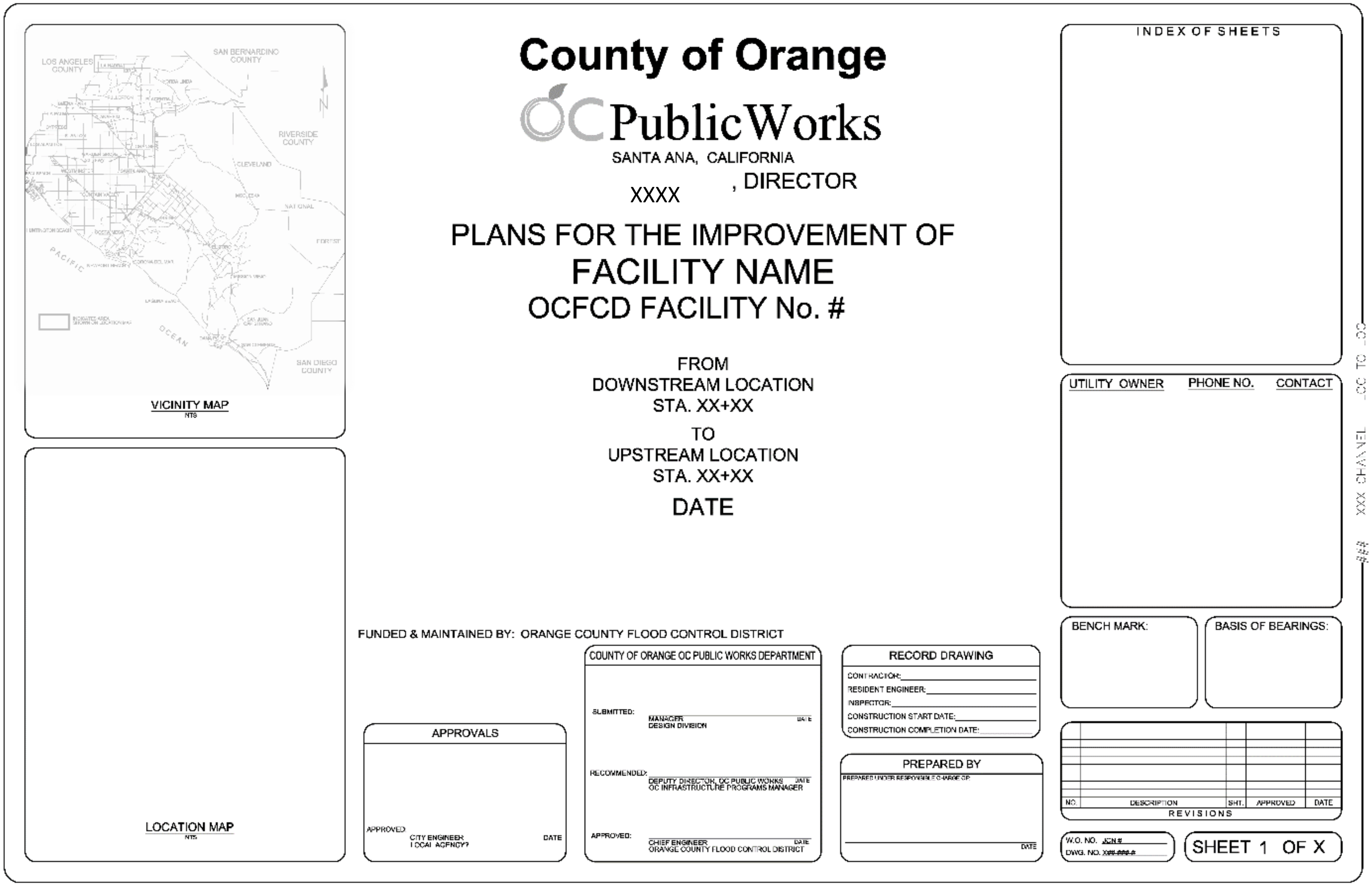



**Figure 2-1: Example Flood Control Title Sheet**

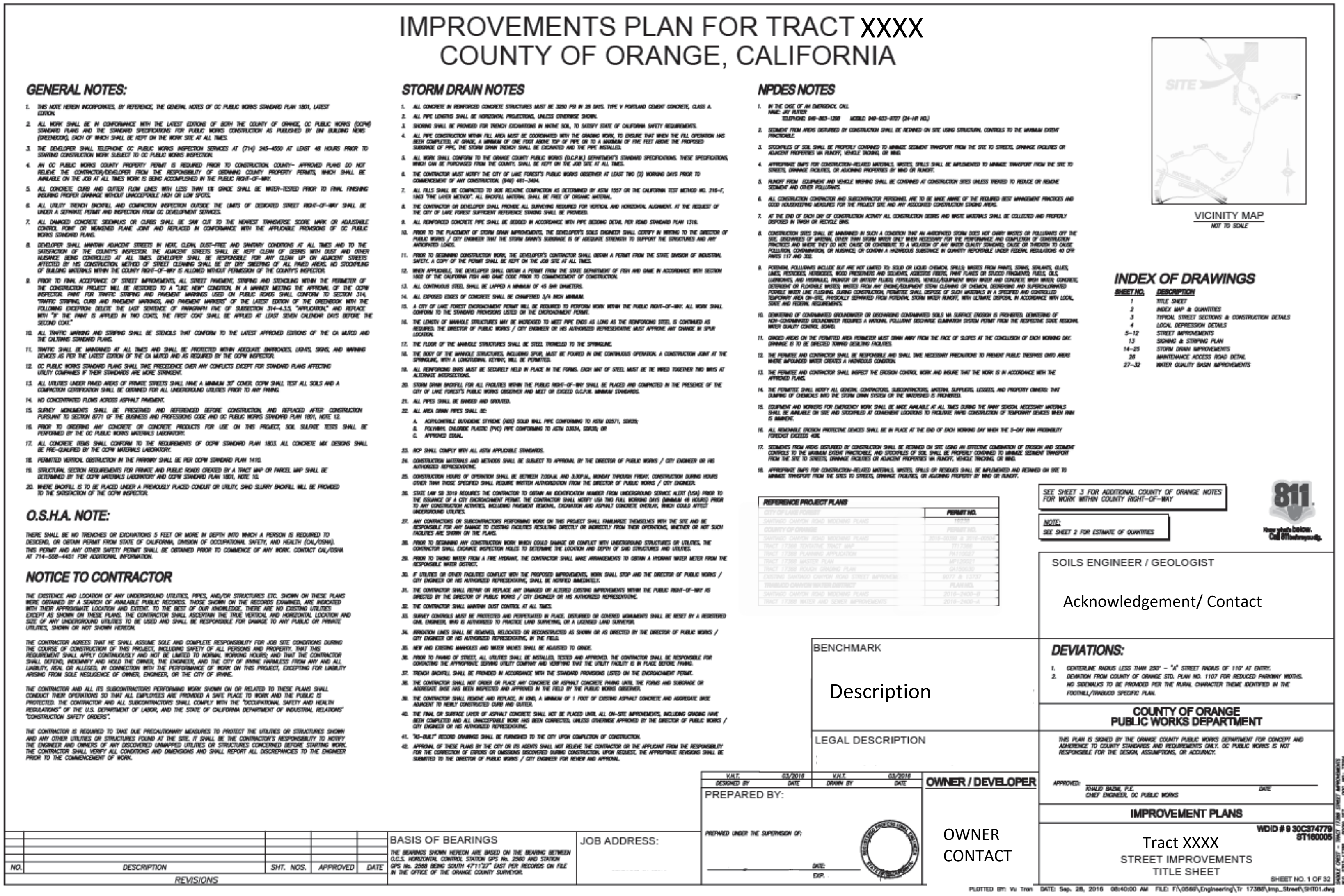



**Figure 2-2: Example Local Drainage Facility Title Sheet**

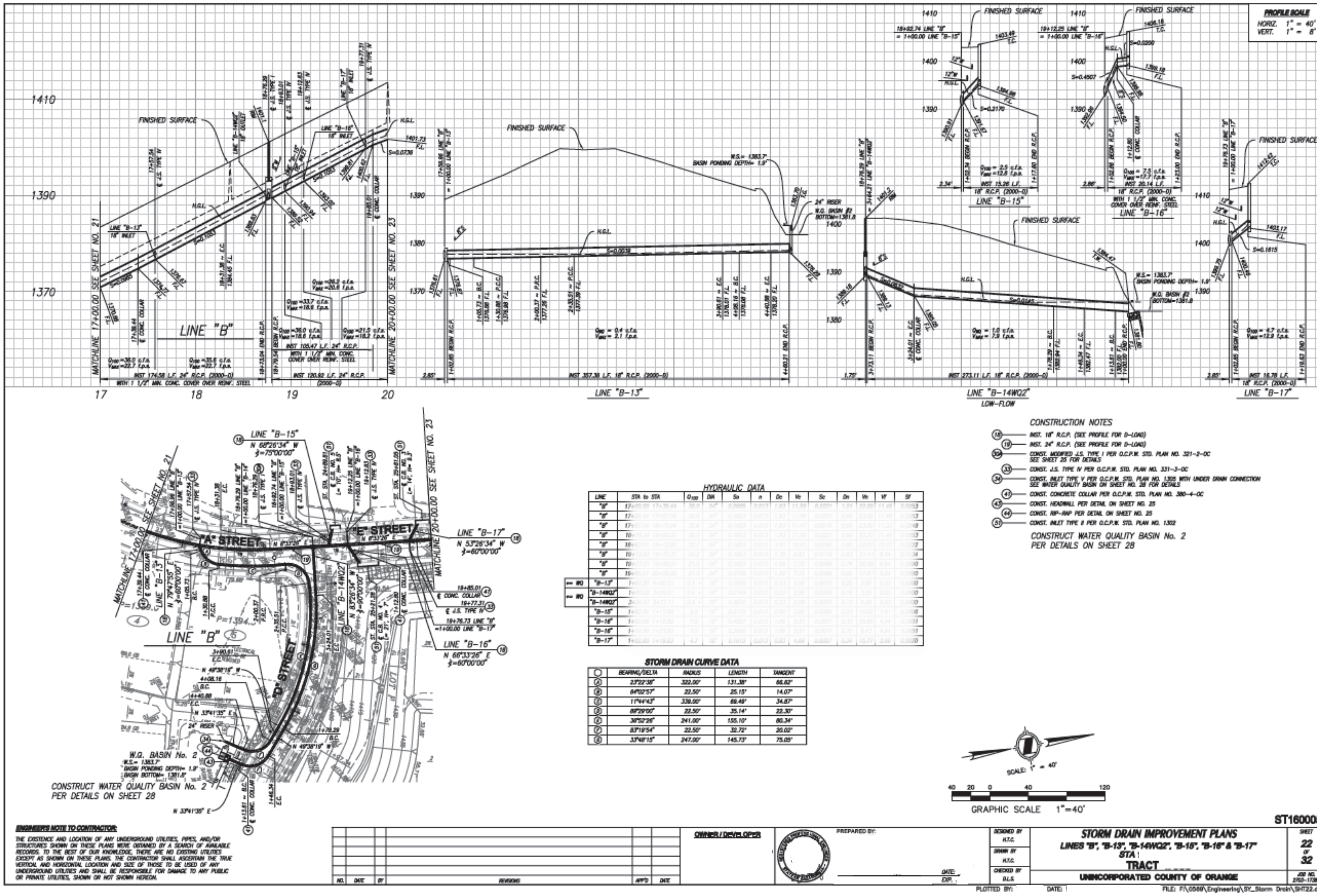


**Figure 2-3: Example Typical Plan and Profile Sheet**

# Chapter 3 Street Drainage and Inlets

## 3.1 Introduction

The storm drain system shall be capable of conveying the storm waters with minimal damage and public inconvenience during the design storm. Presented in this chapter are the criteria and methodologies for design and evaluation of the surface drainage system, which consist of street drainage and storm drain inlets. Although the storm drain/street hydraulics do not specify the actual level of conveyance, the combined effect of all systems should be to provide 100-year protection for all habitable non-flood proofed structures.

## 3.2 Design Criteria

The following design criteria shall be used for storm drain inlet structures built for dedication to the County of Orange, OCFCD, or for private facilities within unincorporated Orange County.

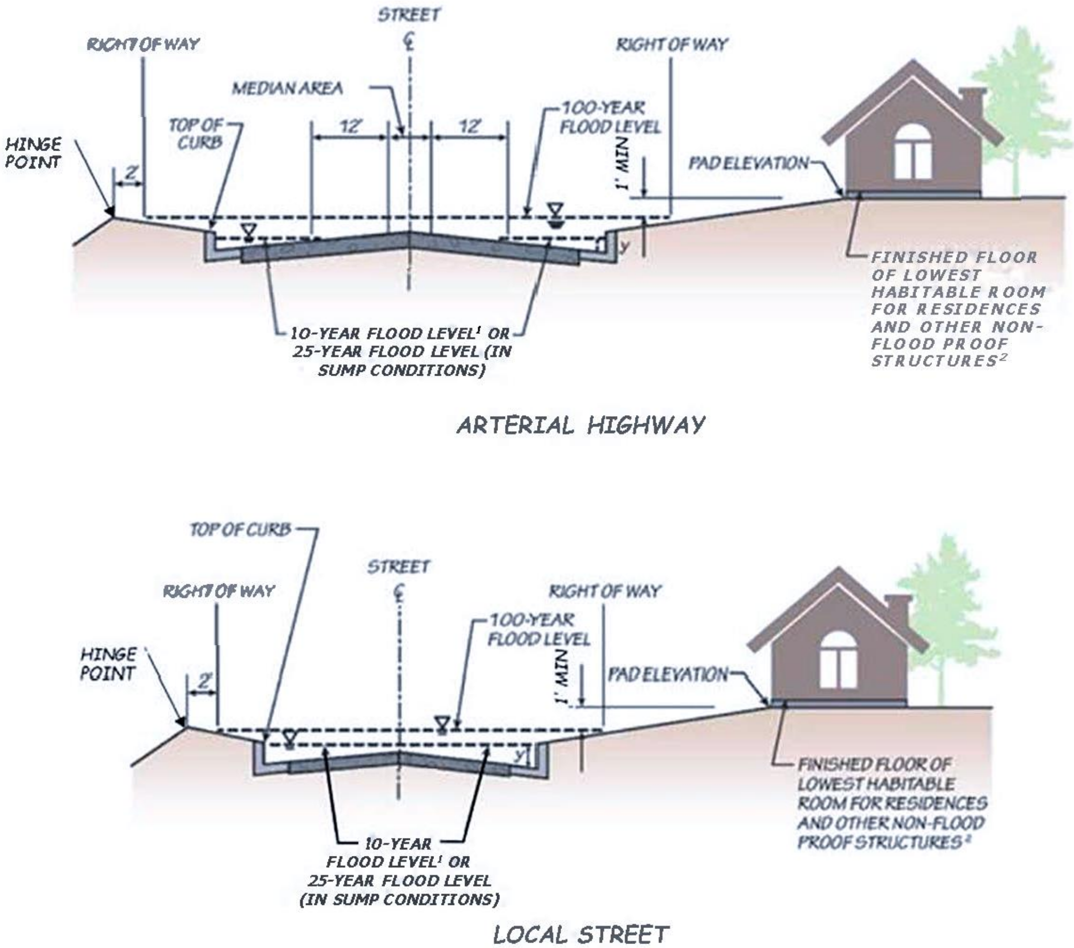
Regional or Sub-Regional design storm frequencies are subject to individual review by OCPW and shall be in accordance with the current Hydrology Manual, its subsequent addendums, and Flood Protection Goals. This manual does not supersede any information contained within the Orange County Drainage Area Management Plan (DAMP) and is intended to be consistent with the DAMP.

The capacity of surface drainage system (street capacity and inlet interception capacity) must be consistent with that of the storm drain conveyance system immediately downstream. The roadway conveyance capacity is constrained by the width, depth, and velocity of flow.

### 3.2.1 Street Drainage

The goal is to provide 100-year protection for all habitable structures pursuant to Public Services and Facilities Element of the General Plan. The 100-YR water surface elevation shall be a minimum of 1 foot below finished floor if a FEMA determined base flood elevation is provided on a Flood Insurance Rate Map, or 1 foot below pad elevation in accordance with Orange County Ordinance No. 09-008 (see Figure 3-1).

Per Figure 3-1, the maximum street flow depth multiplied by velocity (y x v) cannot exceed 6 $ft^2/s$ for pedestrian safety. This is based on experiments that found “A moderate sized person begins to lose stability in three feet of water which is moving at two feet per second”. (FEMA, 1979, pp11). The 10-yr flood level and the 25-yr flood level in sump conditions shall not exceed the top of curb in local streets.




NOTE:
[1]FOR ARTERIAL HIGHWAY, COLLECTOR STREET, AND LOCAL STREET, DEPTH (Y) TIMES VELOCITY CANNOT EXCEED 6 $ft^2/s$.
[2]IF THE FLOOD INSURANCE RATE MAP LISTS A BASE FLOOD ELEVATION (BFE), THEN THE ELEVATION OF THE LOWEST FLOOR OF THE BUILDING, INCLUDING BASEMENTS, OR CELLARS MUST BE AT LEAST 1 FOOT ABOVE THE BFE. If THERE ARE NO BFE', THE BUILDING PAD MUST BE 1 FOOT ABOVE THE CALCULATED 100-YEAR WATER SURFACE ELEVATION FOR NEW DEVELOPMENT.

ORANGE COUNTY PUBLIC WORKS
**Flood Protection Goals**
**Figure 3-1**

**Figure 3-1: Flood Protection Goals**

#### 3.2.1.1 Arterial Highway

Arterial highway design criteria are listed below. Table 3-1 shows the basis of design for different types of roadways.

- One travel lane (use 12' if not determined) shall be free from inundation in each direction in a 10-year storm.
- In a sump condition, one travel lane (use 12' if not determined) shall be free from inundation in each direction in a 25-year storm.
- Median and left-turn pockets shall not be considered as travel lanes.
- In places where super-elevation occurs on arterial highways, an inlet shall be provided (upstream of the point water starts to move across the lanes) as necessary to preclude drainage across the travel lanes. The catch basin shall intercept a minimum of a 10-year storm. Local depressions are not to be used for inlets at medians; grate opening or side opening/grate combination (for which future paving overlap will not create a drop) are recommended. Flooding width from median curbs in super elevated sections shall not exceed 2 feet.
- Flooding must be contained to top of curb for 10-year storm.
- In sump condition, flooding must be contained to top of curb for 25-year storm.
- 100-year flooded width shall not exceed street R/W, unless it can be demonstrated by the applicant that no adverse effect to adjacent low-lying structures is present.
- Cross street flow is not recommended

| Type/Category/Feature | Design Storm | | Design Water Spread | | |
|---|---|---|---|---|---|
| | **25-year** | **10-year** | **Shoulder or Parking Lane** | **½ Outer Lane** | **Local Standard** |
| Arterial-Primary, multilane<br>Speeds over 45 mph | X | | X | | |
| Arterial-Secondary, multilane<br>Speeds 45 mph and under | | X | | X | |
| Low Volume, rural<br>Speeds over 45 mph | X | | X | | |
| Urban<br>Speeds 45 mph and under | | X | | | X |

Notes:
See OCPW Standard Plans for street types.

**Table 3-1: Roadway Design Storm Criteria**

# 3.3 Design Procedure

## 3.3.1 Street Flow

### 3.3.1.1 Hydrology

For calculation of the hydrology for the street drainage, please refer to the current Orange County Hydrology Manual and its subsequent addendums. A link to the online version of the document is included here: OCPW OC Flood Division – Manuals.

### 3.3.1.2 Street Flow Hydraulics Formula

The Street Flow tables (See Appendix A) are based on Standard Curb types A2-6 and A2- 8, rolled curb configurations, and Manning's equation (ignoring the friction along the vertical face as insignificant).

Other configurations will require separate calculations using geometric applications (triangular, trapezoidal, etc.) of the following formula:

$$Q = \frac{1.49}{n}\, A\, R^{2/3}\, S^{1/2}$$

Where:

| | | |
|---|---|---|
| $Q$ | = | discharge (cfs) |
| $n$ | = | Manning's roughness coefficient (dimensionless) |
| $A$ | = | flow area ($ft^2$) |
| $R$ | = | hydraulic radius (ft) |
| $S$ | = | longitudinal gutter slope (not roadway cross slope) (ft/ft) |

### 3.3.1.3 Manning's "n"

Curb to curb "$n$" $= 0.015$ (composite value)

Curb to R/W "$n$" $= 0.030$ (composite value)

"$S$" = longitudinal street slope, not cross fall

#### **3.3.1.4** Standard A2-6 and A2-8 Curb and Gutter

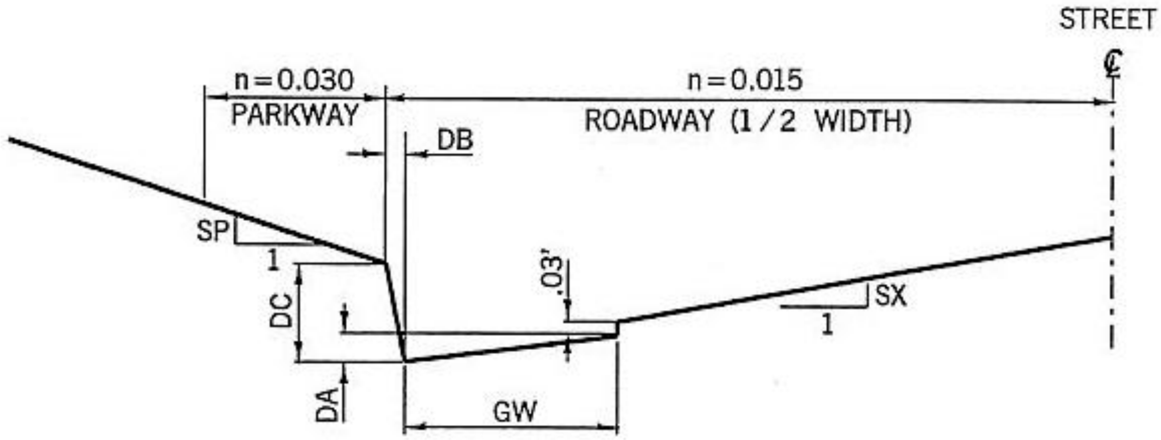



**Figure 3-2: Standard A2-6 and A2-8 Curb and Gutter**

| Curb Type | DA | DB | DC | GW | SP | SX |
|---|---|---|---|---|---|---|
| **A2-6** | 0.125’ | 0.125’ | 0.50’ | 1.375’ | 0.020833’ | 0.017’ |
| **A2-8** | 0.1667’ | 0.1667’ | 0.67’ | 1.833’ | 0.020833’ | 0.017’ |

Notes:

a. For determination of wetted perimeter, vertical depth is used for curb face and horizontal distance is used for gutter, pavement and parkway.

b. Separate area and perimeter calculations are made for parkway and roadway. A composite Manning’s “n” is then used in final conveyance determination in Manning’s equation.

**Table 3-2: Standard A2-6 and A2-8 Curb Dimensions**

#### **3.3.1.5 Standard Rolled Curb and Gutter**

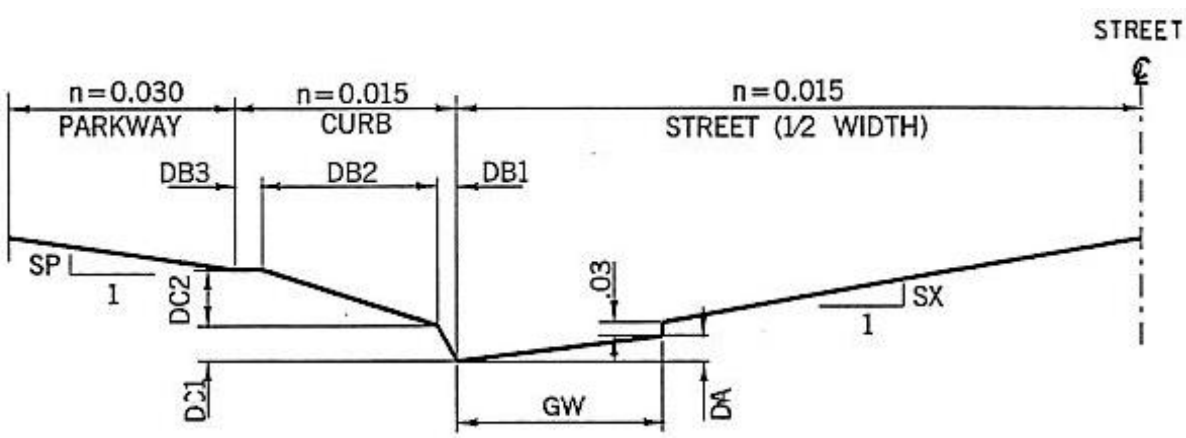



**Figure 3-3: Standard Rolled Curb and Gutter**

| Curb Type | DA | DB1 | DB2 | DB3 | DC1 | DC2 | GW | SP | SX |
|---|---|---|---|---|---|---|---|---|---|
| **Rolled Curb** | 0.125’ | 0.0833’ | 1.79167’ | 0.125’ | 0.0833’ | 0.4167’ | 1’ | 0.020833’ | 0.017’ |

Notes:
a. For determination of wetted perimeter, horizontal distance is used for parkway, gutter and pavement. Actual slope distance is used for curb section.
b. Separate area and perimeter calculations are made for parkway and roadway. A composite Manning’s “n” is then used in final conveyance determination in Manning’s equation.

**Table 3-3: Standard Rolled Curb and Gutter Dimensions**

### 3.3.1.6 Street Flow Tables

The street flow tables were generated to facilitate the engineer’s design and checking of drainage parameters for streets and arterial highways.

There are tables for the standard “vertical face” curb types (A2-6 and A2-8) and for the rolled curb types that are used in Orange County. The tables are color coded, blue for type A2-6 and A2-8 curbs, orange for rolled curbs (Appendix A). The design engineer must use considerable care in selecting a chart, to be sure that the table applies.

The table values included in Appendix A were based upon the following assumptions:

- Curb and Gutter is type “A2-6, A2-8” or rolled curb.
- Manning’s Roughness Coefficient “n” = 0.015 for street and 0.030 for parkway (composite value).
- Roadway cross slope or cross fall $S_x$ = 0.017.
- Values assume triangular flow.
- Values shown are for one-half street, until flow exceeds crown.

Additional Criteria for Local Streets

- Curb and gutter will be type “A2-6”.
- Triangular flow is assumed until crown is exceeded.
- After crown is exceeded, table values are for Full Street.

Raised medians in arterial highways require a special analysis.

#### 3.3.1.6.1 Use of Tables

The tables may be used in five basic ways:

1. The capacity of the half-street may be checked.
2. Splits and routing of flow may be determined.
3. Depth times velocity less than 6 $ft^2/s$ can be checked.
4. Actual conditions of flow may be calculated to size an inlet.
5. Street capacities to street R/W may be calculated.

### 3.3.1.7 Examples

**<u>Capacity of Street Example</u>**

**Given:**

Longitudinal slope of street (S=1%)

Local Street (Street Cross Section, width 32', Type "A2-6" Curb)

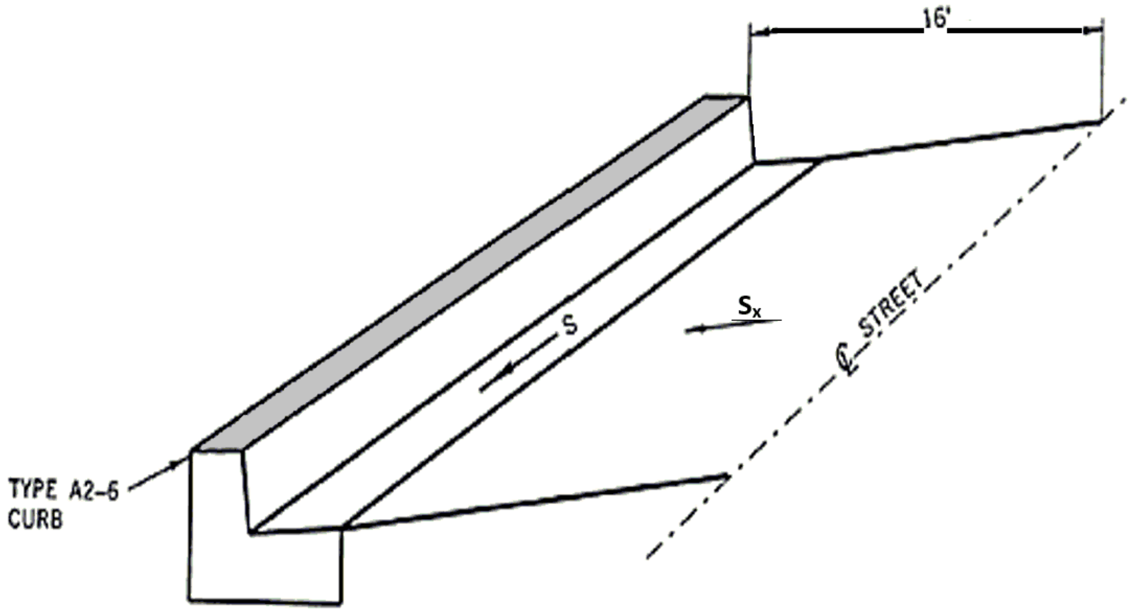



**Figure 3-4: Capacity of Street Example**

**Find:**

Find maximum half street flow (depth should not exceed curb height). Note that the crown height is control for half street flow for most streets.

**Solution:**

See Table 3-4 (a portion of street capacity tables).

Maximum conveyance number is $Q/S^{0.5}$ (discharge divided by square root of the longitudinal street slope) which is 290.0 cfs to top of curb.

$$Q/_{S^{0.5}} = 290.0\ cfs \qquad \text{(slope, } S\text{, in decimal)}$$

$$QMax = 290.0\ (S^{0.5}) = 290(0.01)^{0.5} = 290\ (0.1) = 29.0\ cfs$$

Street Flow Table

Street Half Width = 16’

Curb Type = A2-6”

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.40** | 2.2 | 15.9 | 0 | 0.319 | 59.7 |
| **EXCEEDS CROWN** | | | | | |
| **0.41** | 4.8 | 32 | 0 | 0.28 | 133.4 |
| **0.42** | 5.1 | 32 | 0 | 0.245 | 148.4 |
| **0.43** | 5.5 | 32 | 0 | 0.216 | 164 |
| **0.44** | 5.8 | 32 | 0 | 0.191 | 180.3 |
| **0.45** | 6.1 | 32 | 0 | 0.17 | 197.1 |
| **0.46** | 6.4 | 32 | 0 | 0.152 | 214.6 |
| **0.47** | 6.7 | 32 | 0 | 0.137 | 232.6 |
| **0.48** | 7.1 | 32 | 0 | 0.123 | 251.2 |
| **0.49** | 7.4 | 32 | 0 | 0.112 | 270.3 |
| **0.50** | 7.7 | 32 | 0 | 0.101 | 290 |

Notes:
If street capacity is less than required Q from hydrology report, then an upstream inlet is required.

**Table 3-4: Example Partial Street Flow Table – Street Half Width = 16’, Street Curb = A2-6”**

**Split in Street Example**

**Given:**

$$Q = 10\ CFS$$
$$S^{0.5} = 0.1$$

**Find:**

Find how much flow will be carried by the other side of the street.

**Solution:**

Table 3-4 shows that crown is exceeded. Review of table above shows flow begins to exceed the crown at a conveyance of 59.7 cfs.

$$Q/_{S^{0.5}} = 100$$

$$Q = 59.7\ (S^{0.5}) = 59.7\ (0.1) = 5.97\ cfs$$

Street flow could flow over crown at 5.97 $cfs$.

$$2\ x\ 5.97 = 11.94 \text{ Is larger than } 10\ cfs$$

Therefore, one side of street carries 5.97 $CFS$ and one side carries:

$$10 - 5.97 = 4.03\ CFS$$

Caution must be exercised in split flow assumptions. A better design provides for no split flows.

**Velocity Depth Product ($y\ x\ v$) Example**

**Given:**

Maximum Allowable $y \times v$ product is 6 ft$^2$/s.

**Find:**

$y \times v$

Check the result.

**Solution:**

$$Q/S^{0.5} = 5.97/0.1 = 59.70\ cfs$$

From Table 3-4:

$$\text{Area} = 2.2\ ft^2\ then{:}\ v = 5.97/2.2 = 2.71\ fps$$

From Table 3-4 depth $(y) = 0.40'$

$$y\ x\ v = 0.40\ (2.71) = 1.08 < 6\ ft^2/s$$

**OK**

**OR:**

From the Table "Maximum S" column:

Maximum street slope is $0.319 > 0.01$

**OK**

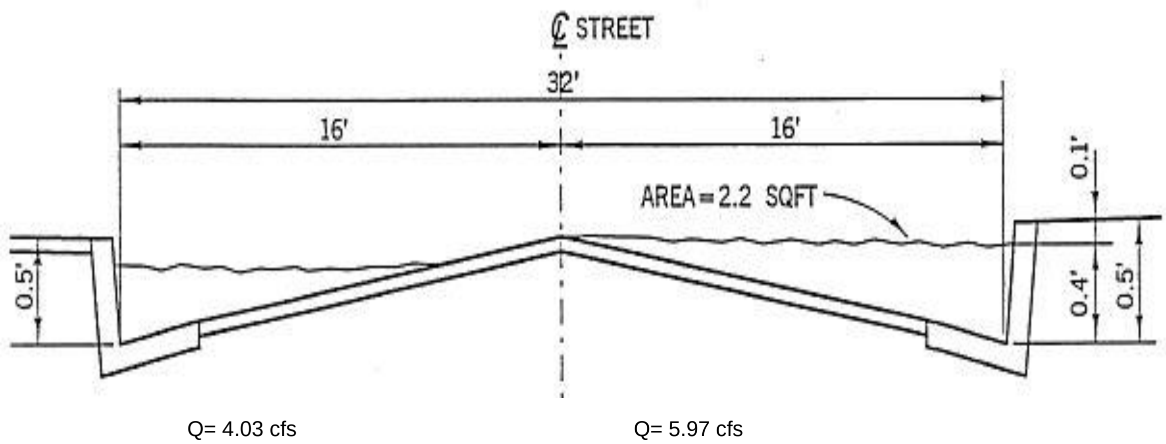



**Figure 3-5: Check Velocity Depth Product Diagram**

Note: The table contains a maximum street slope corresponding to a maximum $y \times v$ product equal to 6 ft$^2$/s. The maximum slope from this table is 0.319 and this is to be used **as a check.**

**Example**

**Given:**

From example above, depth in street is 0.40' which is one of the required inputs into inlet tables.

**R/W Flow Capacity or 100-year Capacity Analysis Example**

**Given:**

Curb type A2-6

Street = 32' (half width 16')

$S$ = 1% = 0.01

**Find:**

Conveyance Factor and Street Capacity

**Solution:**

$$S^{0.5} = 0.1$$

Conveyance factor at R/W is

$$Q/S^{0.5} = 561\ cfs \qquad \text{(Use tables in Appendix A)}$$

Street capacity is

$$(Q/S^{0.5})S^{0.5} = 561.0\ (0.1) = 56.1\ cfs$$

#### 3.3.1.8 Super-Elevation

- General
  The design of transitioning street sections having free water surfaces should include allowances for super-elevation. The usual application is at curb returns.

- Street Super-Elevation
  The inclusion of super-elevation in supercritical street flows should not be construed to mean ready acceptance for this type of design.

  Super-elevation in streets require an inlet to pick up drainage before cross drainage occurs. The inlet should be placed before a zero grade occurs in cross slope of the street section.

### 3.3.2 Inlet Design

#### 3.3.2.1 General

Presented in this section are the criteria and methodologies for design and evaluation of storm drain inlets in Orange County.

There are three types of standard inlets used in Orange County: curb opening, grated, and combination. Additional inlet designs are provided for special installations; these include but are not limited to slotted drains, over slope inlets, and bridge deck inlets.

Inlets are further classified as being on a "continuous grade", "low point", or in a "sump":

- "Continuous grade" refers to an inlet so located that the grade of the street has a continuous slope past the inlet; therefore, ponding does not occur at the inlet.

- "Low point" condition exists whenever water ponds, and the inlet is located at a low point typically on one side of the street. "Low point" flows will breech the crown line of the street before exceeding the top of curb elevations.
- "Sump" condition exists whenever water ponds and the inlet are located at a low point in the street. A sump condition occurs at a change in grade of the street from positive to negative, or at an intersection due to the crown slope of the cross street.

When proposing a sump condition the designer must verify 100-year protection of habitable areas assuming the inlet clogs 100%. Refer to Section 3.3.2.4.2 in this manual for additional discussion.

### 3.3.2.2 Inlet Locations

#### 3.3.2.2.1 Mandatory Inlet Locations

Storm drain inlets must be placed at prescribed locations to protect public safety and provide a minimally functional storm drainage system. These locations include:

- At all corners of arterial highway intersections where flow is directed toward the intersection.
- At low points in street grade, such as sumps.
- Where the flow in the street exceeds top of curb, crown of street, or where bypassed flows are undesirable.
- At intersections where flow is directed towards the intersection without relief from a cross gutter.
- Side inlets are required at end/beginning of super-elevation or other changes in cross-slope of street to prevent excess water from crossing the street.
- Upstream of bridge structure (100% flow intercept required)

#### 3.3.2.2.2 Recommended Inlet Locations

The designer may use engineering judgement to recommend additional inlets at locations in addition to the minimally prescribed locations in order to improve the function of a storm drain system. These locations include:

- Upstream from sump conditions to reduce ponding
- Street intersections to avoid cross gutters
- Point of reduced street grade to prevent sedimentation and to promote safety

#### 3.3.2.2.3 Undesirable Inlet Locations

- Natural drainage courses (see debris considerations in Chapter 9)
- Inlets that require a local depression at median
- Grate inlets should not be used at medians (future paving overlap will create a drop).
- Near ADA ramps

#### 3.3.2.2.4 Street Slope at Inlet

It is common practice to place inlets at the downstream end of street grade changes. This allows the designer to use the higher depths developed by the reduced longitudinal grades in the street. The designer should be aware that the water depths change gradually due to momentum, therefore consideration of the street grade and location of the inlets must be considered.

In order to diminish the effects of momentum, the inlet must be placed a minimum distance downstream of the end of the vertical curve, as shown in Figure 3-6.

### 3.3.2.3 Specific Criteria for Private Drainage Systems

- The grate type inlets may be substituted for curb inlets if street grade does not exceed 5%.
- If street slope exceeds 5%, side inlet catch basins with local depressions shall be used.
- Design shall provide for minimum number of cross gutters (to reduce splashing).
- Street nuisance flow shall be contained in standard curb and gutter or a 4’ wide concrete alley gutter.
- Curb drains (Grading Section requires minimum 4” drains) shall have an opening 3” less curb height where occurring in public R/W.

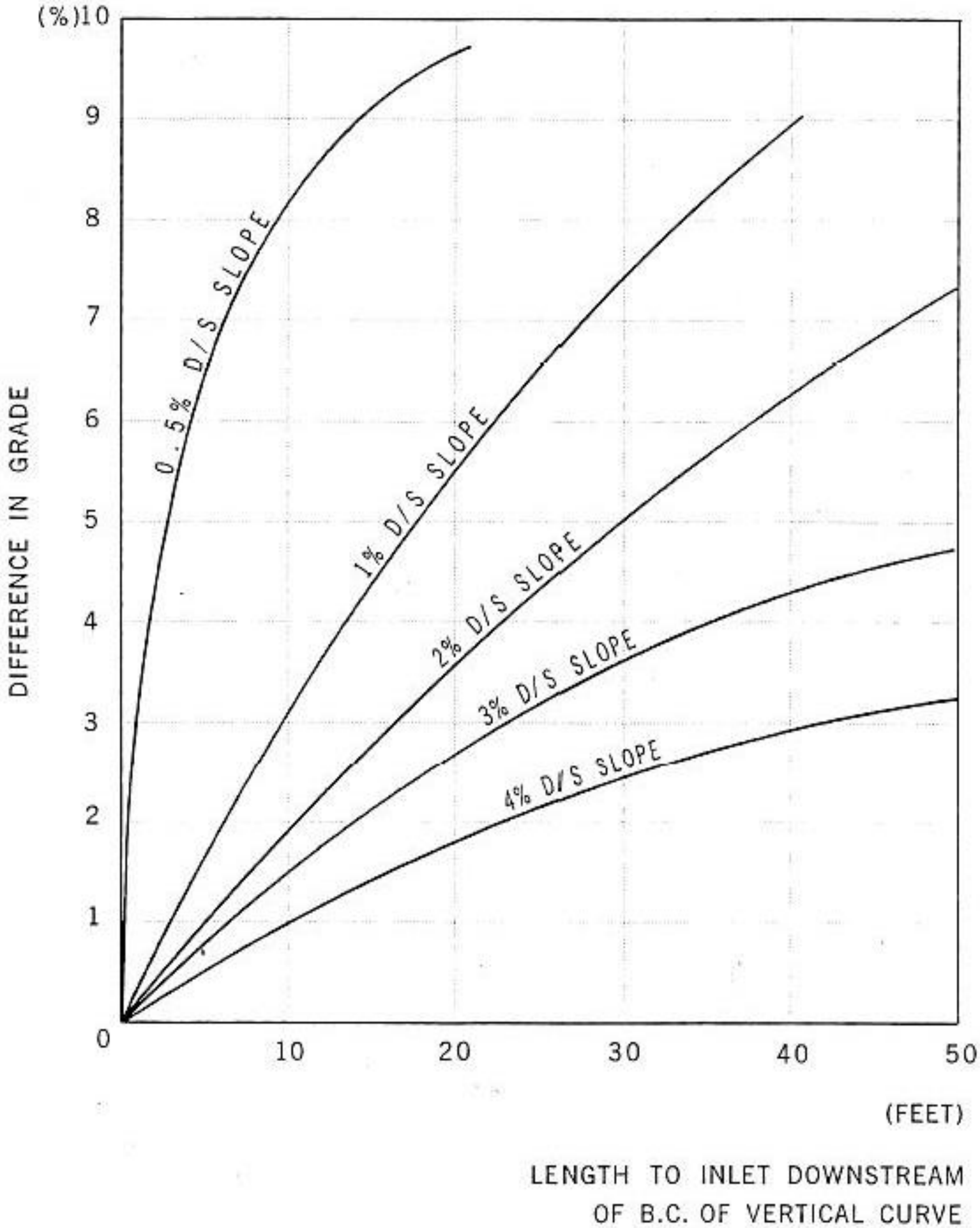



**Figure 3-6: Length to Inlet Downstream of Vertical Curve**

### 3.3.2.4 Inlet Capacity

#### 3.3.2.4.1 Capacity of Inlets on Continuous Grade

The length of a curb opening inlet may be decreased by allowing part of flow to pass the opening. A maximum of fifteen percent (15%) is recommended to be bypassed. The bypassed flow must be included in the capacity calculations for the next downstream inlet, whether it be on continuous grade or within a sump.

#### 3.3.2.4.2 Capacity of Inlets in Sumps

When proposing a sump condition the designer must verify 100-year protection of habitable areas assuming the inlet clogs 100%.

This will require a secondary emergency outlet for the sump waters, which should provide a minimum of 1.0’ freeboard between the maximum water surface elevation and the minimum finish floor elevation. This emergency outlet system must direct overflows to either a downstream street with adequate capacity or a natural conveyance system. Point of discharge must be analyzed with regard to prevention of downstream problems. Such a system need not consist of additional structures, but may require modification of surrounding grading, allowing water to flow between dwelling units.

### 3.3.2.5 Limit of Series Inlets

Mainline storm drains should not flow through any inlets due to clogging potential. If more than two inlets in series are needed, a lateral drain should be provided. No more than two inlets in series may be present on any storm drain lateral.

### 3.3.2.6 Curb Opening Inlets

#### 3.3.2.6.1 General

The standard inlet in Orange County uses the curved faceplate and therefore has a high hydraulic efficiency when depths exceed hydraulic opening as shown in Table 3-4. The use of the square faceplate reduces efficiency and should be used with caution when street flow depth approaches top of opening (the orifice equation applies.)

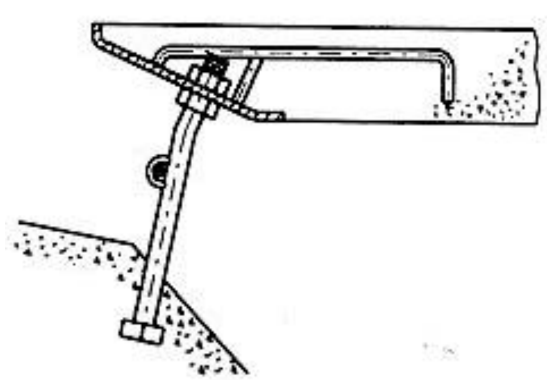


**Figure 3-7: County of Orange Curved Face Plate**

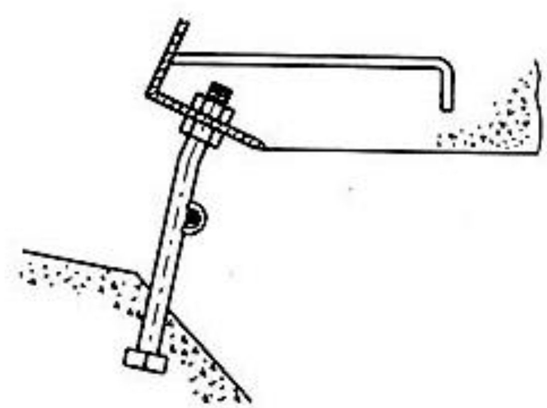


**Figure 3-8: County of Orange Square Face Plate**

The inlet hydraulic tables contained herein are applicable to both cases of faceplates if the depth of water (y) does not reach the faceplate. See Table 3-5 for hydraulic opening. It should be noted that all hydraulic calculations for curb opening inlets in this section assume a non-restricting clear opening area and a 6” free flowing nappe. Any type of lattice covering over inlet openings need consideration for possible clogging or validity of curb opening interception formulas due to exit flow conditions being restricted.

| Curb Height | Hydraulic Opening | |
|---|---|---|
| | Curved- Face-Plate | Square Face-Plate |
| 4” | 5.7” | 4” |
| 6” | 7.5” | 6” |
| 8” | 9.3” | 7.9” |

**Table 3-5: Height of Opening for OCPW Standard Curb Openings with 4” Local Depressions**

### 3.3.2.6.2 Standard Lengths of Curb Inlets

Standard OCPW inlet lengths are 3.5’, 7’, 14’, and 21’. Field practice has demonstrated that a separation between inlets is needed once the combined inlet length exceeds 21 feet. Minimum separation between inlets shall be 12 feet.

### 3.3.2.6.3 Reduction of Inlet Width

The inside width dimension of catch basins, as shown on the Standard Plans, may be reduced to avoid conflicts with structures or utilities. The reduction in the width dimension that will be necessary shall be determined by the designer, but in no case shall the inside width be less than 30 inches.

### 3.3.2.6.4 Capacity of Curb Opening Inlets with Partial Interception

#### 3.3.2.6.4.1 General

For the “continuous grade” condition, the capacity of the inlet is dependent upon many factors including gutter slope, depth of flow in the gutter, heights and length of curb opening, street cross-slope, and the depth of depression at the catch basin. In addition, not all of the gutter flow will generally be intercepted, and some flow will continue past the inlet area (“bypass”). The amount of bypass must be included in the drainage facility evaluation as well as in the design of the inlet.

Design procedure:

1. Calculate the street flow to the most upstream catch basin.
2. Calculate the amount of flow intercepted by the upstream catch basin.
3. If intercepted flow is less than the street flow, carryover the difference between the street flow and the intercepted flow to the next downstream catch basin.
4. Calculate the street flow for the next downstream catch basin and add the carryover flow from the upstream basin to the street flow from the downstream basin to determine the total flow at the downstream catch basin.
5. Calculate the amount of flow intercepted by the downstream catch basin.
6. If intercepted flow is less than the total flow, then bypass the difference between the total flow and the intercepted flow to the next downstream catch basin.
7. Repeat as needed.

#### 3.3.2.6.4.2 Depth of Water at Inlet

The depth of the water at the inlet's entrance for a given discharge shall be calculated by an iterative procedure using the Federal Highway Administration's (FHWA's) formula (Hydraulic Engineering Circular No. 22 (HEC-22) Research Report 1267-1F, Third Edition, University of Texas at Austin as noted in section 4.3.2.2 of said circular using the following parameters:

- $S_x$ = Cross slope of the pavement at the curb
- $S_W$ = Cross slope of the local depression or gutter. AC hinge point to Inlet FL (see Figure 3-9)
- $n$ = Manning's coefficient
- $S$ = Longitudinal slope of the gutter

#### 3.3.2.6.4.3 Capacity of Standard Inlet

The Length ($L_T$) of a curb opening inlet (with flow depths below the faceplate) intercepting 100 percent of the flow in the gutter is given by FHWA's formula per HEC-22:

$$L_T = K_U\, Q^{0.42}\, S_L^{0.03}\, [1/(n\, S_e)]^{0.6}$$

Where:

| | | |
|---|---|---|
| $K_U$ | = | 0.6 |
| $L_T$ | = | Curb opening length required to intercept 100% of the gutter flow (ft) |
| $S_L$ | = | Longitudinal slope (ft/ft) |
| $Q$ | = | Gutter flow ($ft^3/s$) |
| $S_e$ | = | Equivalent street cross slope (ft/ft) |

The standard curb opening inlet includes a depressed gutter section or local depression. The equivalent cross slope, $S_e$ can be computed using:

$$S_e = S_X + S'_W\, E_0$$

Where:

| | | |
|---|---|---|
| $S_X$ | = | Street cross slope (ft/ft) |
| $S'_W$ | = | Cross slope of the gutter measured from the cross slope of the pavement ($S_X$) |
| $S'_W$ | = | $S_W - S_X$ |
| $S_W$ | = | Cross slope of local depression or gutter. AC hinge point to Inlet FL (ft/ft) |
| $S_W$ | = | $a/W + S_X$ |
| W | = | Width of local depression or gutter (ft/ft) |
| a | = | Gutter Depression-Difference in street cross slope projection and inlet flowline (in) |
| $E_0$ | = | Ratio of flow in local depression to total gutter flow determined by the gutter configuration upstream of the inlet. |

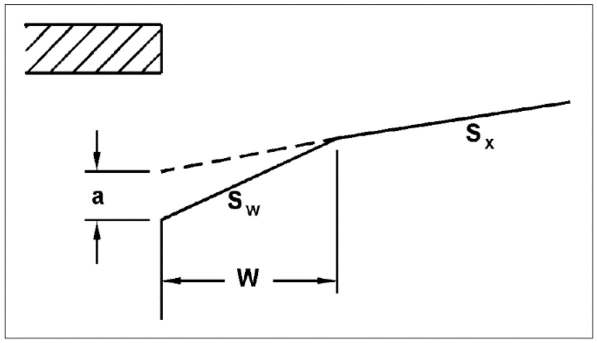



**Figure 3-9: Definition Figure for Inlets**

| Commonly Used Local Depression Dimensions | | | | | | | |
|---|---|---|---|---|---|---|---|
| Curb Type | Curb Height (in) | Local Dep. Drop (in) | Local Dep. W (ft) | Total H to Top of Curb (in) | $S_x$ (ft/ft) | FHWA a (in) | FHWA $S_w$ (ft/ft) |
| A2-6 | 6.0 | 1.5 | 1.5 | 7.5 | 0.0170 | 3.07 | 0.1875 |
| A2-6 | 6.0 | 4.0 | 4.0 | 10.0 | 0.0170 | 5.57 | 0.1330 |
| A2-8 | 8.0 | 2.0 | 2.0 | 10.0 | 0.0170 | 3.97 | 0.1823 |
| A2-8 | 8.0 | 4.0 | 4.0 | 12.0 | 0.0170 | 5.97 | 0.1413 |

**Table 3-6: Local Depression Dimensions**

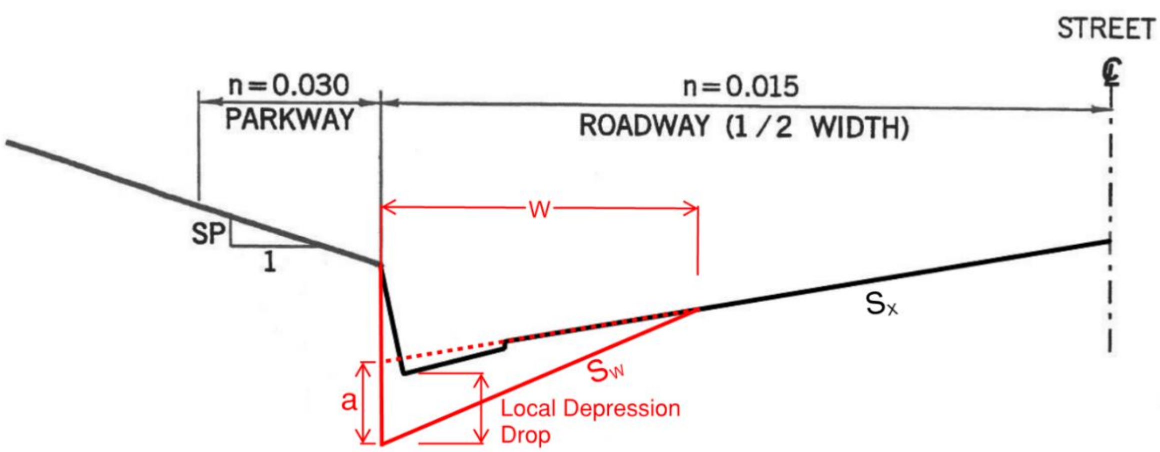



**Figure 3-10: Typical FHWA Curb Opening Inlet Cross Section**

The efficiency (E) of a curb-opening inlet shorter than the length required for total interception ($L_T$) is expressed by:

$E=1-[1-(L/L_T)]^{1.8}$

Where:

L = Proposed curb-opening length (ft)

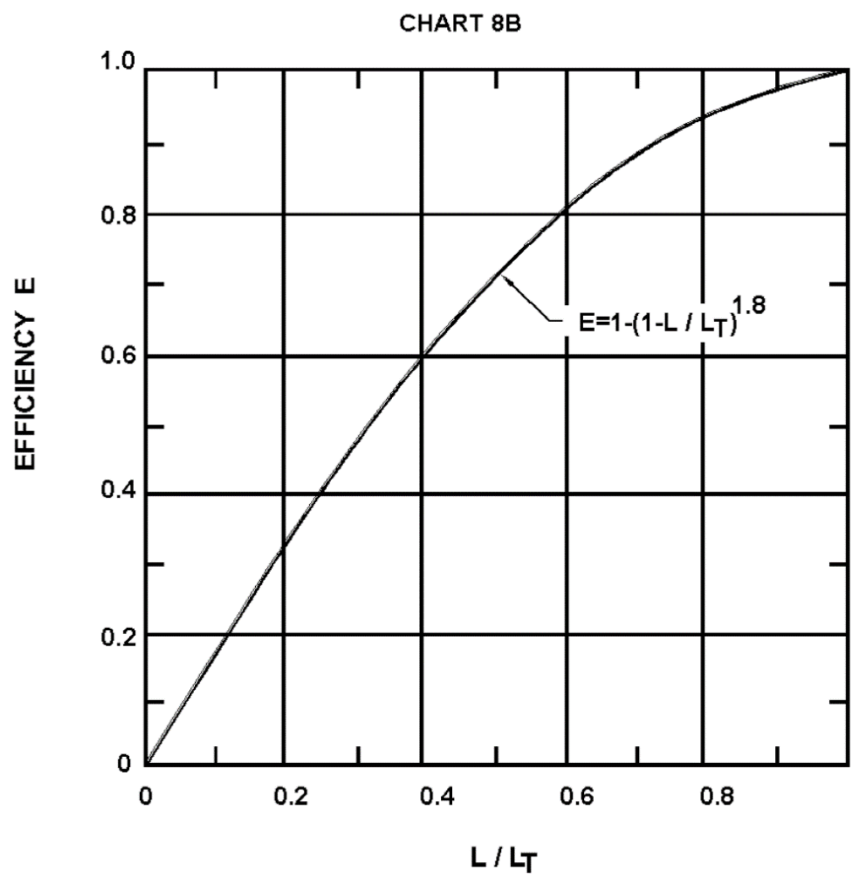



**Figure 3-11: Curb Opening Inlet Interception Efficiency**

#### 3.3.2.6.4.4 Sizing Length of Inlet

To size an opening length the following information must be known:

- Height of the curb opening ($h$)
- Design flow ($Q$) in the gutter (drainage area, rainfall intensity and runoff coefficients are included in the hydrology design discharge analysis). Any carryover from a previous inlet must be included.
- Depth of flow ($y$) for the longitudinal slope ($S$) and composite street section including the local depression at the inlet in question. This will be determined using the FHWA formulas and associated tables in HEC-22.

#### 3.3.2.6.4.5 FHWA HEC-22 Software

FHWA Hydraulic Toolbox version 4.4 dated July 2018 or current version is a free calculating software that incorporates the street and curb-opening formulas per FHWA's HEC-22 that were noted previously and can be used for County or Orange and OCFCD projects. The use of this program will assist the designer with time saving solutions. Also, the "Local Depression" feature shall not be used as directed by FHWA at this time and should be set to zero.

#### 3.3.2.6.4.6 Previous Inlet Sizing Formula

For new developments, the previously accepted 1950 Izzard curb opening inlet sizing formula for continuous grade conditions will no longer be accepted by the County of Orange or OCFCD due to more currently accepted inlet sizing formula per FHWA HEC-22 for curb opening inlets. Any updates to the FHWA HEC-22 formulas for curb opening inlets in a continuous longitudinal slope grade will be incorporated into this manual.

For retrofits to existing development, with less than two inlets affected, the 1950 Izzard curb opening inlet sizing formula may be used at the discretion of County of Orange or OCFCD. See Appendix E for Izzard method used in 1996 LDM.

### 3.3.2.6.5 Design Procedure for Continuous Grade

The capacity and length of a curb opening inlet may be decreased by allowing part of flow to pass the opening. A maximum of 15% is recommended to be bypassed.

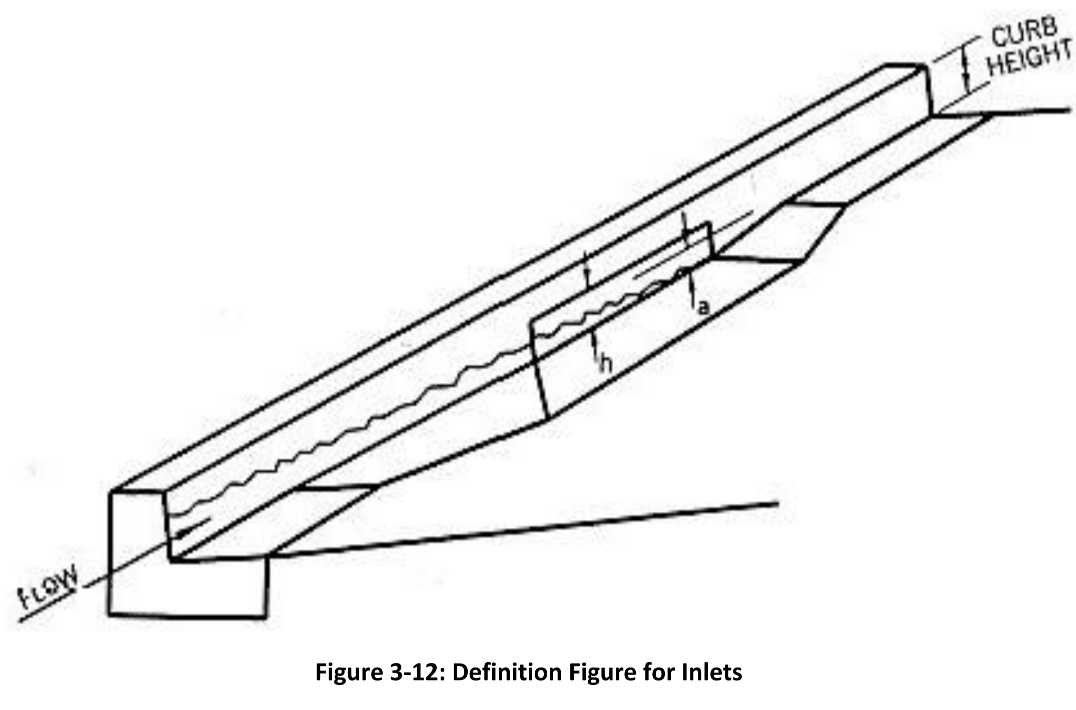



**Figure 3-12: Definition Figure for Inlets**

Design procedure:

1. Determine $Q$ to inlet.

For the following steps use of FHWA's Hydraulic Toolbox 4.4 or current version is recommended.

2. Determine depth of flow $(y)$ in the street composite cross section including the local depression (see Table 3-6) using FHWA formulas and associated tables in HEC-22. Ignore AC lip-up.
3. Determine best design of inlet to use, checking that the calculated depth is less than the height of the curb opening per Table 3-2, Table 3-3, & Table 3-5.
4. Determine length of inlet $L_T$ required to intercept 100% of the gutter flow using the FHWA formulas and associated tables in HEC-22.
5. Using proposed L and calculated $L_T$, determine Efficiency E, the proportion of the total gutter flow intercepted by the inlet in question which shall be a minimum of 0.85.
6. The partial flow intercepted, $Q_p$,is the total gutter flow $Q$ times E .
7. The flow bypassed in the street to the next inlet, $Q_b$, is then $Q - Q_p$. Return to #1.

#### 3.3.2.6.6 Example:

**Given:**

$Q$ in street = 6 cfs

Longitudinal Slope *S* = 0.01

Curb height = 8”

Local depression drop = 4”

Local depression width W = 4’

**Find:**

y, the depth of water at curb

**Solution:**

$S_W$ = 0.1413 (Table 3-6)

Use Standard curved face inlet (see Figure 3-7)

Using an iterative process or Hydraulic Toolbox 4.4: Depth of water at curb, $y$ = 6.88”

From Table 3-2: h = 9.3” height of curb opening

Check:

$$h > y$$

9.3” > 6.88” **OK**

From calculation, charts or Toolbox 4.4 (E=1.0): Length of catch basin for 100% interception:

$$L_T = 12.8'$$

Propose 10' long catch basin:

**Use $L$=10'**

$$L/L_T = 10/12.8 = 0.78$$

From Figure 3-11 or Toolbox 4.4: Efficiency:

$$E = 0.93$$

$$Q_b = E \times Q = 0.93 \times 6.0 = 5.6\ cfs\ (bypass\ to\ next\ CB)$$

Repeat the calculation procedure for the next catch basin.

Figure 3-13 shows the inlet definitions for this example.

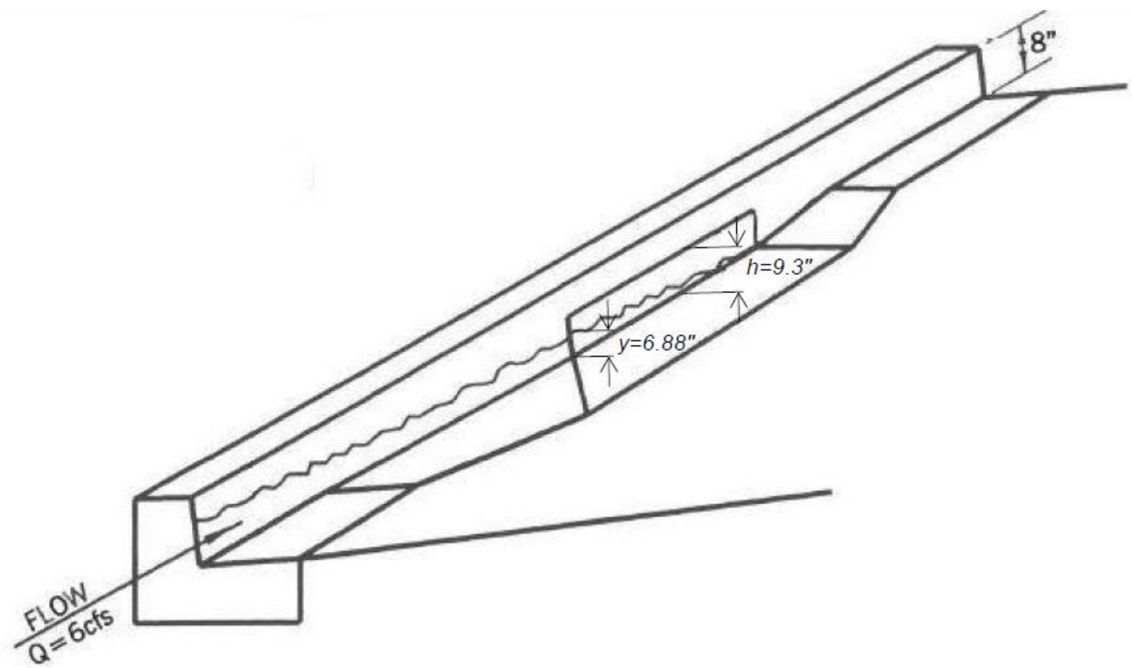



**Figure 3-13: Inlet Definition**

See Figure 3-14 for the FHWA Hydraulic Toolbox 4.4 solution:

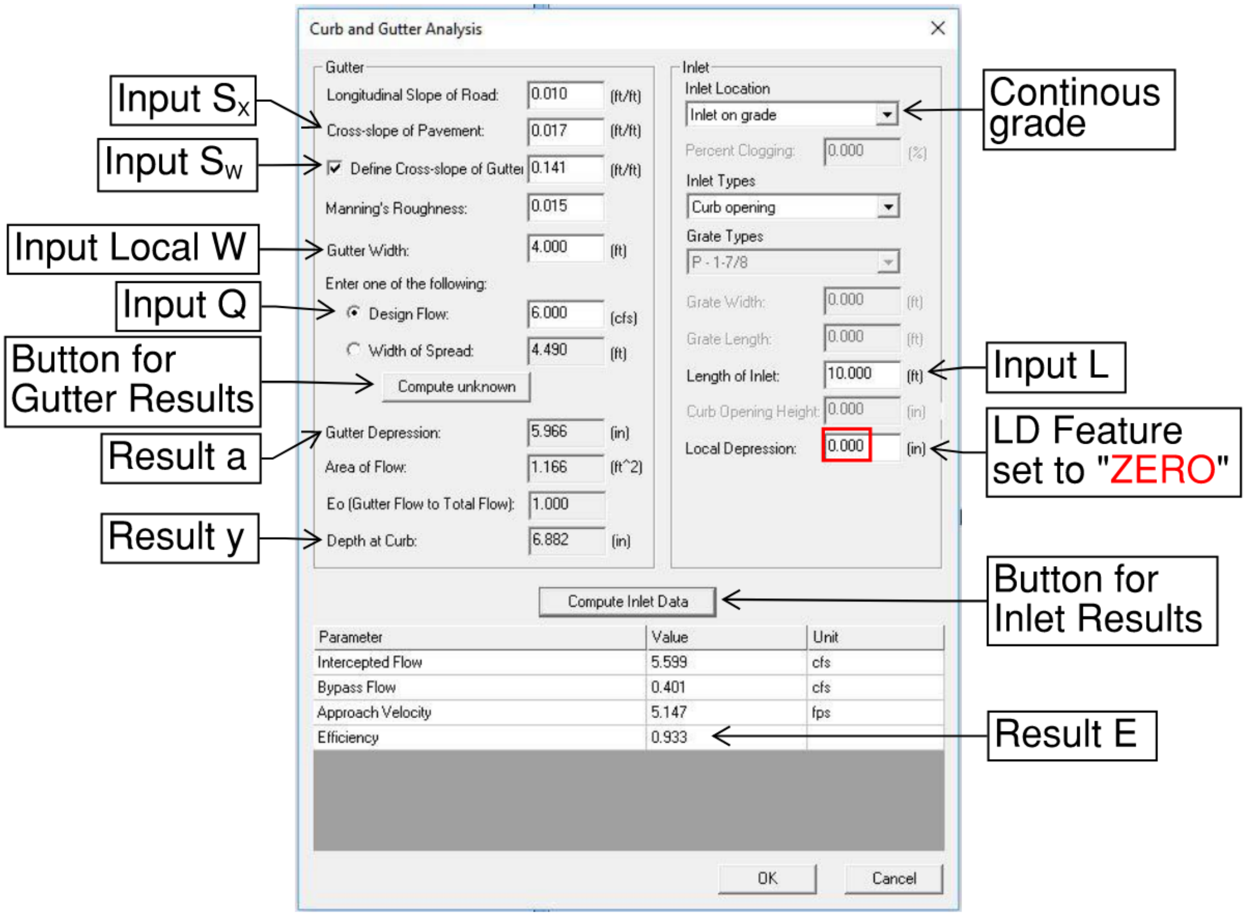



**Figure 3-14: Example FHWA Hydraulic Toolbox 4.4 Solution**

### 3.3.2.7 Standard Calculation Format

A standard form calculation format has been included as Figure 3-15.

Page: ____ of ____

Designer: ________________

Location/Street: ________________

Date: ________________

Inlet: ______________________________________________

Curb Opening (Interception)

Given:

(a) Discharge Q ____ = ______ cfs

(b) Street Slope S = ________ ft/ft

(c) Type of Curb A2-____ "D" Other ______

(d) Half Street Width W = ______ ft

(e) Street Cross Slope $S_x$ = ________ ft/ft

(f) Manning's Roughness n = ________

(g) Local Depression Width w = ______ ft

(h) Local Depression Drop Drop = ______ in

(i) Local Depression Cross Slope $S_w$ = ________ ft/ft (see Table 3-6)

(j) CB Clear Opening Height h = ______ in (see Table 3-5)

Solution:

Depth in street y = ______ in (Verify y > h)

Width of Spread T = ______ ft

Ratio Flow on LD/Gutter to Total $E_o$ = ______

CB Length for 100% Interception $L_T$ = ______ ft

Iteration:

Proposed CB Length L = ______ ft

$L/L_T$ = ______ / ______ = ______ ft/ft

Efficiency E = ______ (see Figure 3-9 or Toolbox)

$Q_p = QxE$ = ______ x ______ = ______ ft/ft (*Intercepted)*

$Q_b = Q\text{-}Q_p$ = ______ - ______ = ______ ft/ft (*Bypass to* ______)

**Figure 3-15: Curb Opening Calculation Form**

### 3.3.2.7.1 Capacity of Curb Opening Inlets in a Low Point or Sump

The capacity of a curb opening inlet in a sump or low point varies with the length of the inlet (L) and the depth of water at the entrance (H = a + y). The inlet will operate as a weir until the water submerges the entrance. When the depth of water is about 2 times the height of the entrance or more, it will operate as an orifice. Between these two depths the inlet will operate somewhere between a weir and orifice.

### 3.3.2.7.2 Nomograph Parameters (see Figure 3-16)

- Physical basis of Nomograph
  - The curb opening inlet may be located on a continuous grade or at a low point in the grade. The low point is created by depressed gutter on continuous grade street.
  - All flow coming to the inlet must eventually enter the inlet and will pond until enough head is built up so the flow through the inlet will equal the peak inflow from the gutters.
- The hydraulic basis of the nomograph is as follows:
  - For heads (depth of water) less than the height of the opening (i.e., H/h less than 1.0), the inlet acts as a weir with the flow passing through critical depth at the entrance per the formula:

$$Q = 3.087\ LH^{3/2}$$

Note:
This formula depends on a 6” free nappe.  Any hydraulic conditions that lessen this freeboard requirement will need to analyze the validity of this formula.

- For hydraulic depth with $H/h$ between 1 and 2 a transition was used since the operation of the inlet is not defined.
- For hydraulic depth equal to or greater than twice the height of opening (i.e., $H/h$ greater than 2) the inlet acts as an orifice per the formula.

$$Q = 5.62\ h^{3/2}\ (H'/h)^{1/2}L$$

Where:

$H'$= equal to the hydraulic depth on the middle of the inlet opening ($H' = H - h/2$)

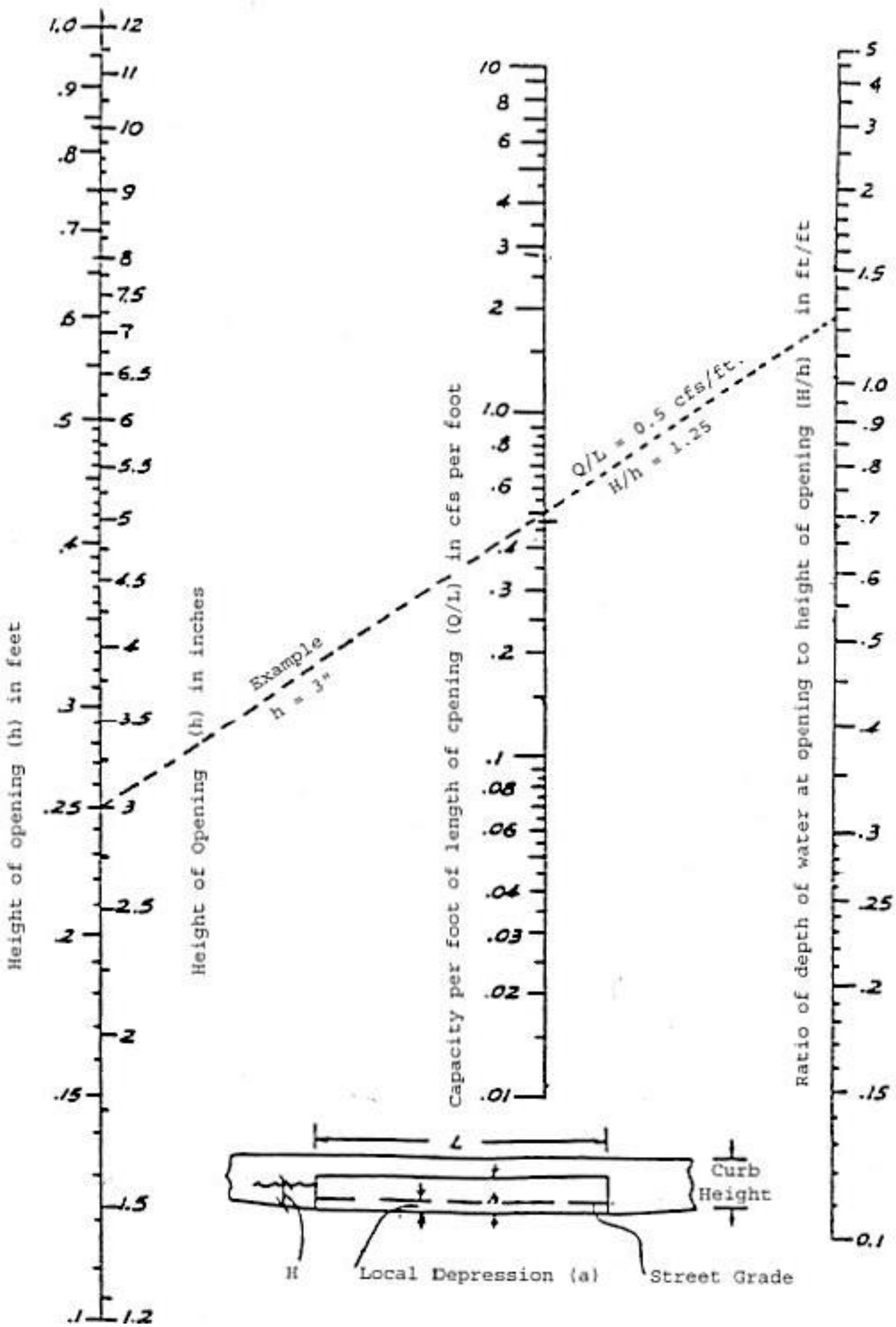



**Figure 3-16: Capacity Nomograph at Curb Opening Inlets at Low Points**

**Page:** ________ of ________
**Designer:** ________________
**Location/Street:** ________________
**Date:** ________________

**Inlet:** ______________________________________

**Curb Opening (Sump)**

**Plan Sketch**

**Given:**

(a) Discharge, Q: ________ = ________ cfs
(b) Type of Curb Inlet:

A-2 - ________ "D" 4" Rolled 6" Rolled

Other: ______________________________

**Solution:**

H (depth at opening): ________ inches
h (height at opening): ________ inches
H/h = ________ / ________ = ________

SEE Figure 3-17: Capacity Nomograph at Curb Opening Inlets at Low Point

Q/L = cfs/ft *from Nomograph*

$L_{required}$= Q/(Q/L) = ________ / ________ = ________ feet

L = ________ feet **<-Use**

**Secondary Overflow/Release Location:** ______________________________

**Figure 3-17: Sump Calculation Form**

### 3.3.2.8 Grate Type Inlets

#### 3.3.2.8.1 General

Use of grate type inlets in sumps within streets is prohibited. Grated inlets are covered by OCPW Standard Plan 1304. The main considerations in hydraulic design of grated inlets are the geometry of the grate, width of street flooding and the flow through areas of the openings.

Inlet grates act as a strainer, catching debris that obstructs the grate openings. However, safety design requires closely spaced bars and precludes increased spacing of the bars. Following are general grate/bicycle design criteria considerations:

- Openings shall consist of at least 50% of total area of the grate.
- Grated inlets are not recommended in bicycle lanes.
- For grates within bicycle lanes, bicycle safety crossbars shall be provided at a maximum spacing of 9” perpendicular to direction of travel (a 24” diameter bicycle wheel will not drop down more than about 1”).
- Minimum clear spacing between longitudinal bars shall be 1”.
- Grates shall be cast-iron or galvanized steel.

#### 3.3.2.8.2 Grate Inlets on a Continuous Grade

##### 3.3.2.8.2.1 General

Where the street gutter is on a continuous grade, grate inlets shall not be assumed to intercept all stormwater.

- The efficiency of a grate inlet, where the gutter is on a continuous grade, increases when part of the flow is allowed to go past the inlet. This is due to the increased depth (head) in the cross-section of flow over the grating.
- Street flow is usually not directed to a grate, as there is no local depression. This condition allows for interception of only flow over the grate.
- A curb opening placed upstream from the grate on a continuous grade tends to take off debris brought down as flow begins, thus reducing the probability of the grate becoming clogged.

##### 3.3.2.8.2.2 Configuration

- The bars shall run parallel to the direction of flow.
- The unobstructed opening parallel to the direction of flow (required to allow the jet of water falling through the opening at the downstream end of the slot) depends upon the depth and velocity of flow in the approach gutter and the thickness of the grate. The minimum length of the slot may be estimated by the following empirical formula:

$$L_{min} = 0.675v\ (y + t)$$

Where:

| | | |
|---|---|---|
| $L_{min}$ | = | minimum length of slot (ft) |
| $v$ | = | mean velocity of flow in the approach gutter (fps) |
| $y$ | = | depth of water in approach gutter (ft) |
| $t$ | = | thickness or depth of grate (ft) |

#### 3.3.2.8.2.3 Design Procedure for Grate Only Inlets

1. Determine depth of flow in street using street capacity charts.

   $Y$ = Depth of flow determined from street capacity charts.
2. Determine capacity of grate using Figure 3-18, Figure 3-19, or Figure 3-20 depending upon grate length.

   Note: Hydraulic irregularities caused by I-beam in Figure 3-19.

#### 3.3.2.8.2.4 Grate Inlet Capacity Curves

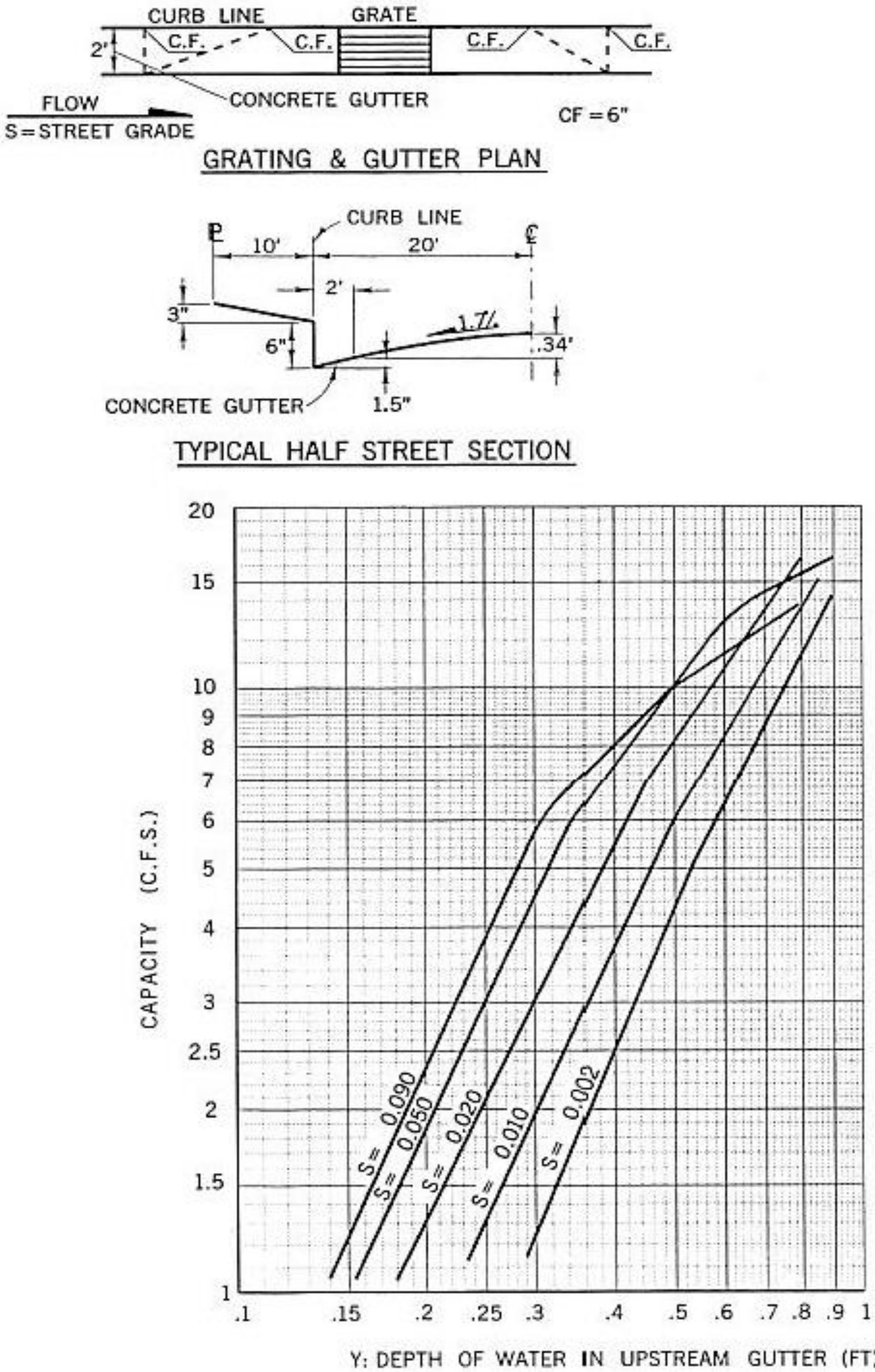
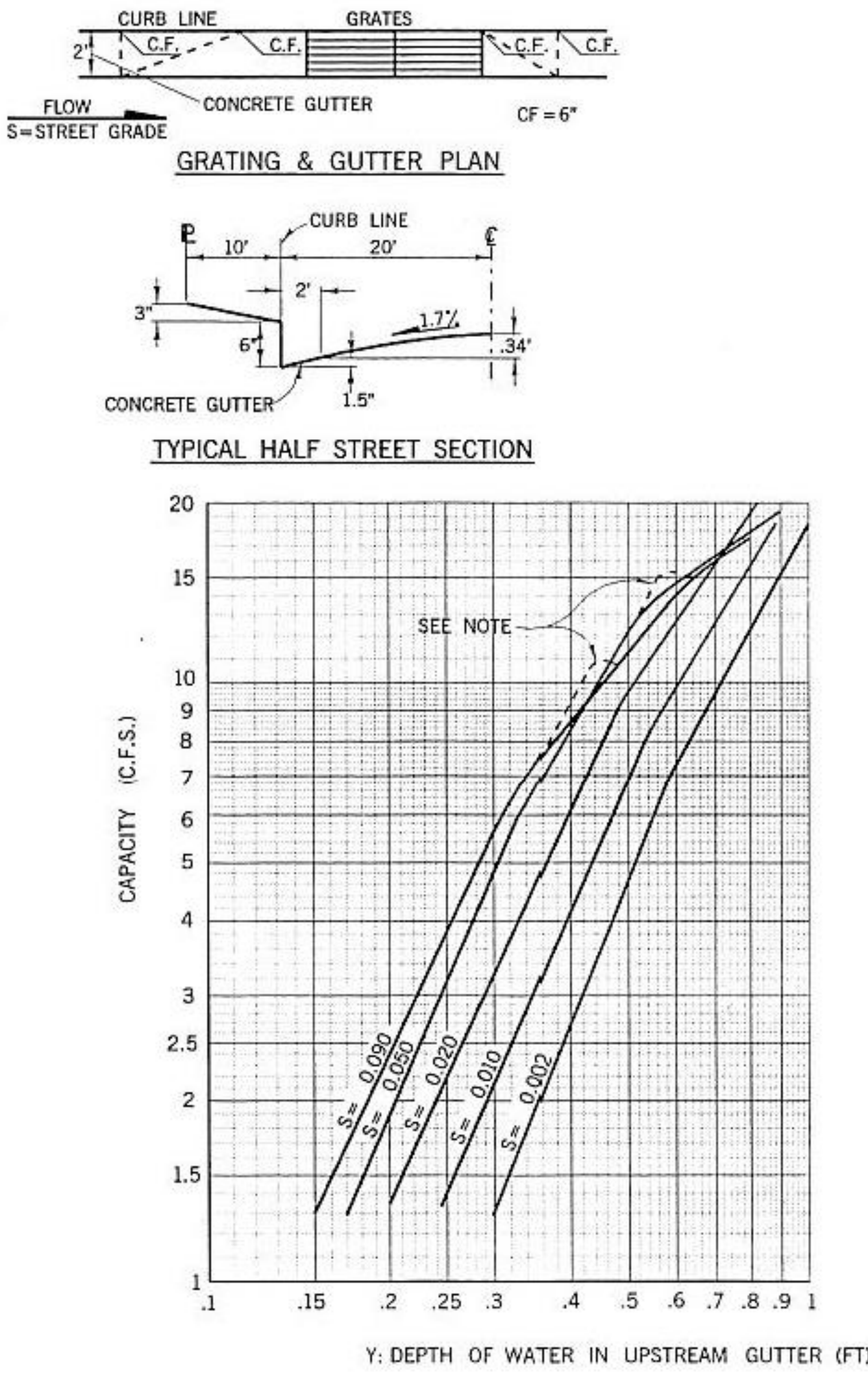



**Figure 3-18: Grated Inlet Capacity (Length = 2'-10⅞")**




Note: Hydraulic Irregularities Caused By I Beam

**Figure 3-19: Grated Inlet Capacity (Length = 6'-2⅞")**

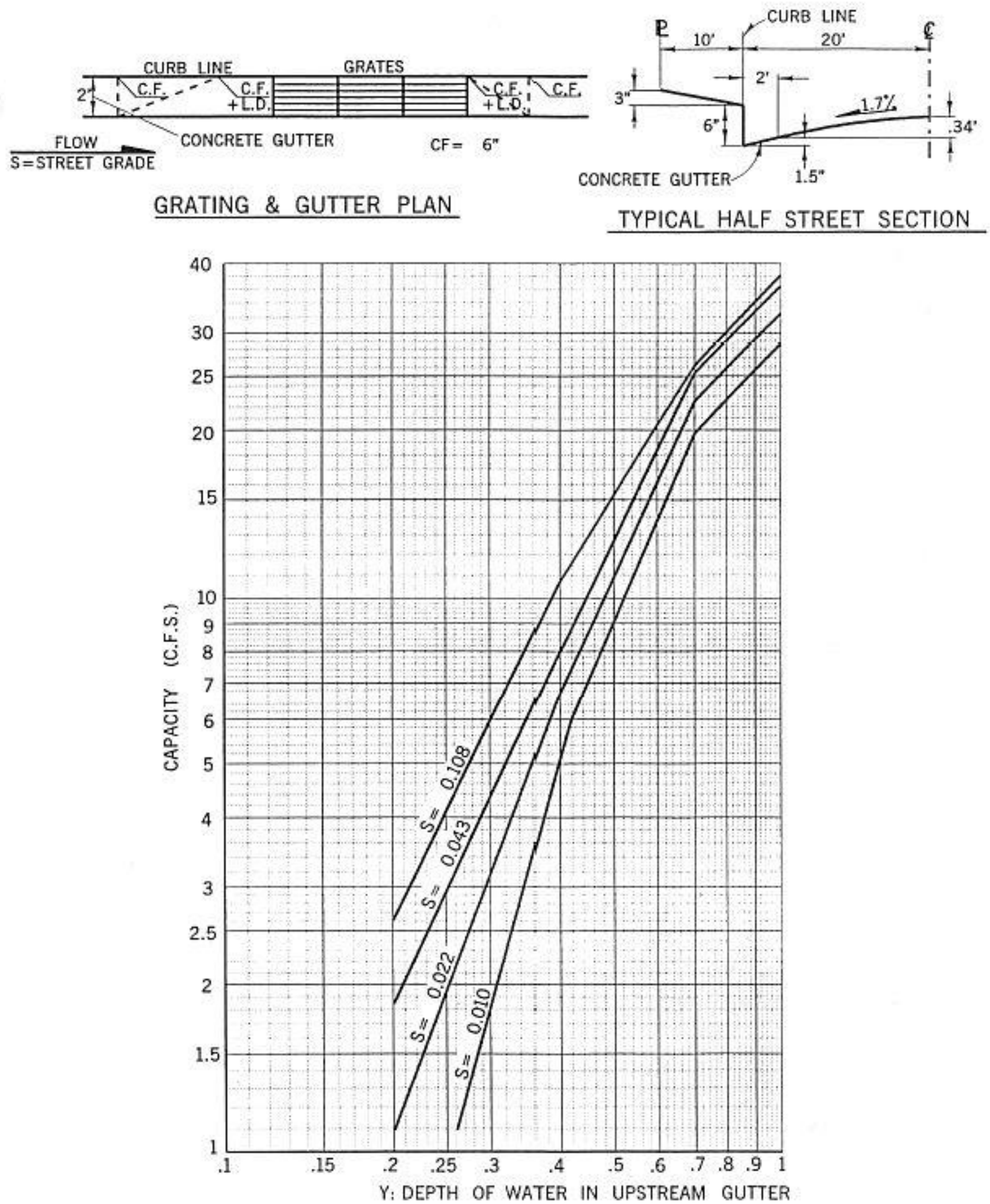



**Figure 3-20: Grated Inlet Capacity (Length = 9'-6⅞")**

#### 3.3.2.8.3 Grate Inlets at a Sump

Use of grate inlets in sumps within streets is prohibited.

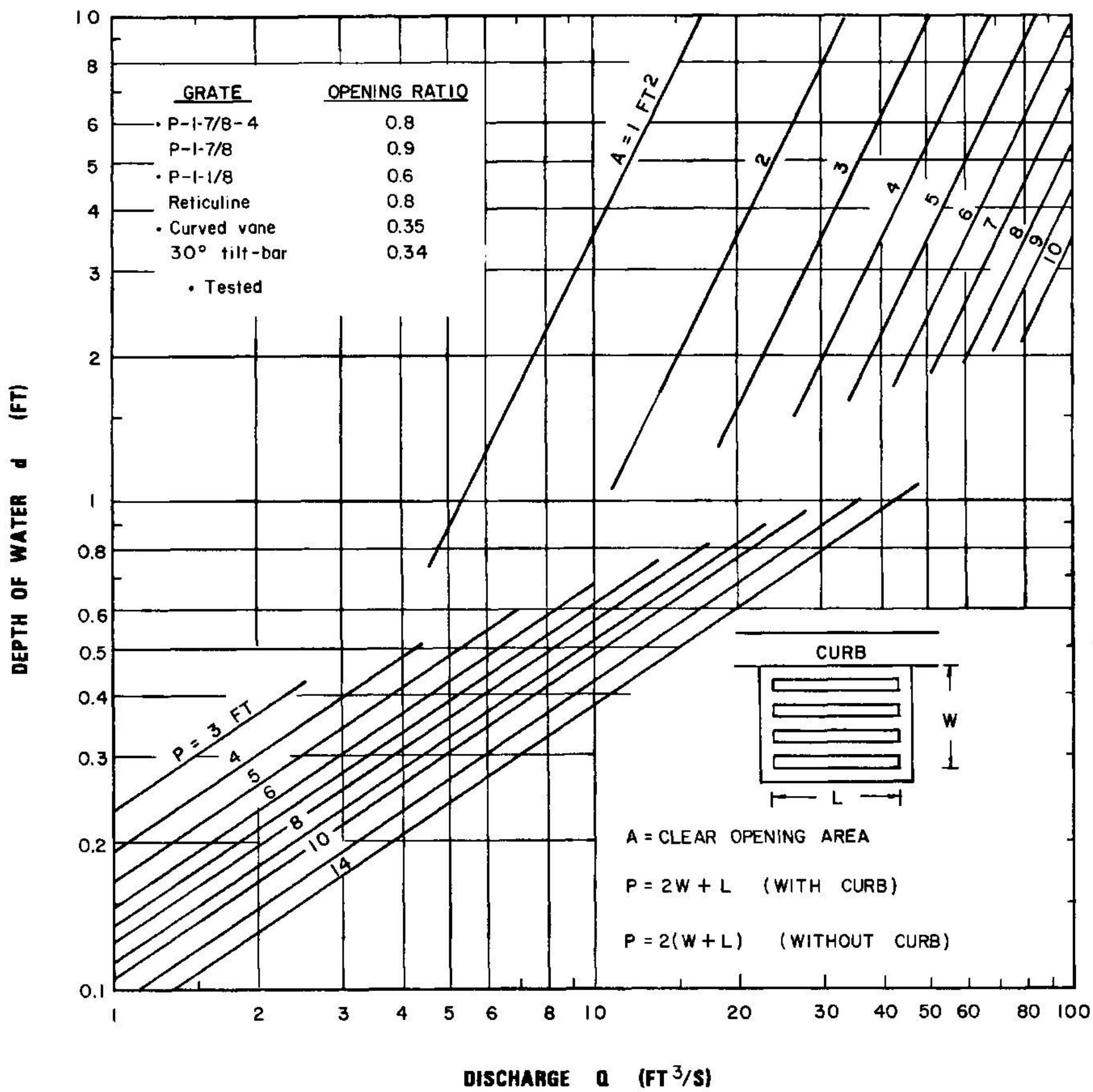
The capacity of the grate depends upon the area of the openings and the depth of the water at the grate. Empirical data suggests that a grate will act as a weir and follow the weir formula for depths (heads) on the grate up to 0.4 ft. It will act as an orifice and follow the orifice formula for heads of 1.4 ft and over. For heads between 0.4 ft and 1.4 ft, the operation is not defined because of vortices and eddies over the grate.

When proposing a sump condition, the designer must verify 100-year protection of habitable areas assuming the grate clogs 100%. This will require a secondary emergency outlet for the sump waters, which should provide a minimum of 1.0-foot freeboard between maximum water surface elevation and minimum finish floor elevation. This secondary emergency outlet system must direct overflows to either a downstream street with adequate capacity or a natural conveyance system. Point of discharge must be analyzed with regard to prevention of downstream problems. The emergency outlet system needs not consist of additional structures but may require modification of surrounding grading to allow water to flow between dwelling units.

##### 3.3.2.8.3.1 Design Procedure for Sump Grates

- Typically, the following data are given.
    - Design discharge ($Q$)
    - Grate configuration (adjacent to curb or in open area)
- Assume grate dimensions and include grate details with calculations.
- Compute the perimeter of the grate opening, $P$, ignoring the bars and omitting any side over which the water does not enter, such as when one side is against the face of a curb. Divide the result by 2. This allows for partial clogging of the grate by assuming that only half of the perimeter will be effective.
- Compute the total area of clear opening, $A$, excluding area taken up by bars and divide by 2. This allows for partial clogging of the grate by assuming that only half of the area will be effective.
- Enter Q on Figure 3-21 using the design discharge.
- If design discharge intersects appropriate $P$ curve, then read the required head at the left margin. In this case, the grate perimeter is the control. This is the usual case.
- If design discharge does not intersect appropriate $P$ curve, then find the intersection of design discharge and appropriate $A$ curve and read the required head at the left margin. In this case, the grate area is the control.

# CHART 9B




Grate Inlet Capacity in Sump Conditions - English Units

**Figure 3-21: Grate Inlet Capacity in Sump Conditions Assuming No Clogging (HEC 22) (Brown, et al., 2009)**

#### 3.3.2.8.3.2 Detailing Information

The inlet floor shall be sloped 3:1 (H:V) toward the outlet. In a shallow drain system where conservation of head is essential, or any system where the preservation of a non-silting velocity is necessary, the half-round floor shown in Figure 3-22 should be used.

Empirical results have shown the "vane" type L and V grates (shown in Figure 3-23) will accept more water than any of the conventional grate styles under virtually all flow conditions. Even with extremely high volume and velocity conditions in the gutter, very little, if any, water will not be captured by the grate, provided the water passes over it. In addition to its increased capacity, the vane grate is also bicycle safe. For these reasons, it is the most desirable (hydraulic) style available.

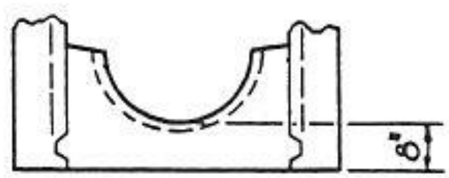



**Figure 3-22: Half Round Inlet Floor**

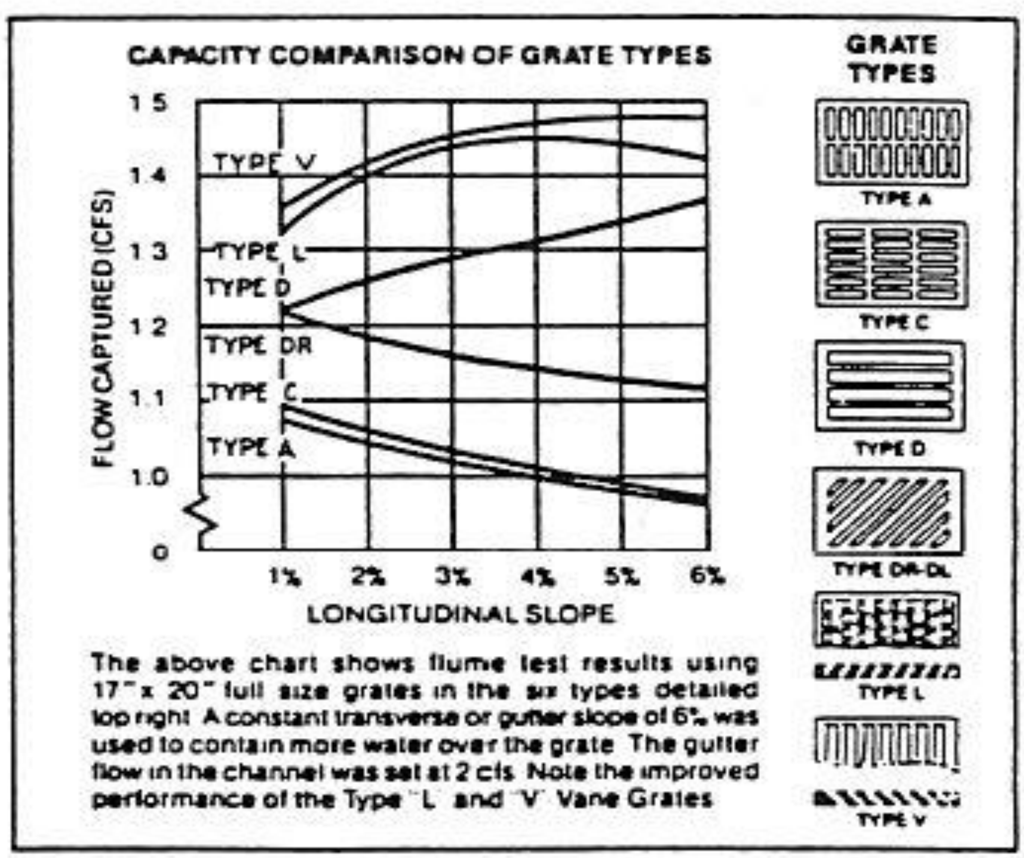
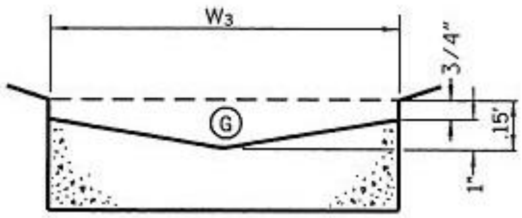



**Figure 3-23: Capacity Comparison of Inlet Grate Types**




GIVEN

$Z_1$ = left cross slope
$Z_2$ = right cross slope
$S$ = street slope
$Q$ = discharge

ASSUME

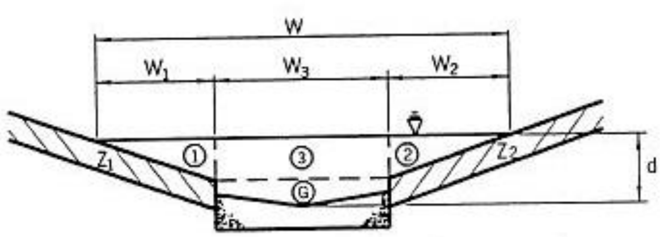
$d$ = depth of flow

GUTTER

Gutter area = $AG = \frac{.08W_3}{2} + .06\ W3 = 0.10\ W3$

WETTED
PERIMETER = $PG = \left(2\sqrt{(0.08)^2 + \left(\frac{W_3}{2}\right)^2}\right) + 0.13$

$d_{full} = 0.15'$




CALCULATE

1. $W = W_1 + W_2 + W_3$

   $W_1 = \frac{d - 0.15}{Z_1}$

   $W_2 = \frac{d - 0.15}{Z_2}$

   $W_3$ = width of gutter

2. $P = P_1 + P_2 + P_G$

   $P_1 = \sqrt{(d - 0.15)^2 + W_1^2}$

   $P_2 = \sqrt{(d - 0.15)^2 + W_2^2}$

3. $A = A_1 + A_2 + A_3 + A_G$

   $A_1 = \frac{(d - 0.15)\ W_1}{2}$

   $A_2 = \frac{(d - 0.15)\ W_2}{2}$

   $A_3 = (d - 0.15)\ W_3$

4. $R = A/P$

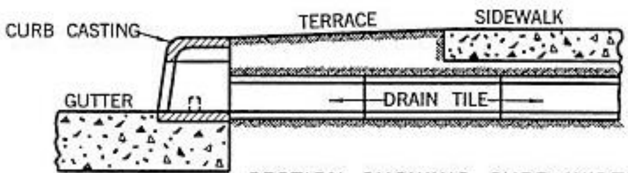
5. $V = \frac{1.49}{N}\ R^{2/3}\ S^{1/2}$

CHECK

$Q = AV$

ALLEY CAPACITY FORMULAS FOR DETERMINATION OF DEPTH OF WATER OVER GRATE INLETS

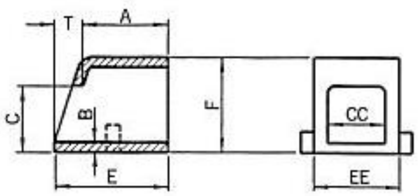
**Figure 3-24: Grate Gutter Example**




SECTION SHOWING CURB WIDTH

| CATALOG NO. | DIMENSIONS IN INCHES | | | | | | | | WT. LBS |
|---|---|---|---|---|---|---|---|---|---|
| | A | B | C | CC | E | EE | F | T | |
| R-3262-1 | 5 | 1/2 | 4 | 4 | 6 | 5 | 7 | 1 | 20 |
| R-3262-2 | 6 | 1/2 | 4 | 4 | 8 1/4 | 5 | 6 | 2 1/4 | 20 |
| R-3262-3 | 5 | 1/2 | 4 | 5 1/2 | 6 | 1/2 | 6 1/2 | 1 | 20 |
| R-3262-4 | 5 | 1/2 | 4 | 16 | 7 | 17 | 6 3/8 | 2 | 45 |
| R-3262-5 | 6 | 1/2 | 5 | 5 | 8 | 6 1/8 | 8 | 2 | 30 |
| R-3262-6 | 6 | 1/2 | 6 | 6 | 8 | 7 1/4 | 9 | 2 | 30 |

NEENAH FOUNDRY

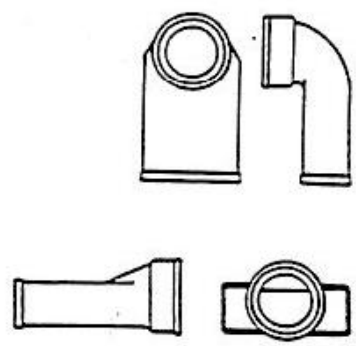


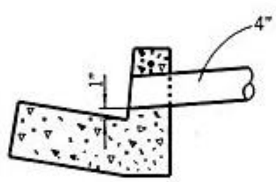

ADAPTORS – ROUND TO RECTANGULAR PIPE

| ROUND PIPE | RECT. PIPE | LAYING LENGTH |
|---|---|---|
| * 4" | 3"X5" | 1'–0" |
| * 5" | 3"X9" | 1'–0" |
| 6" | 3"X12" | 1'–0" |
| * 7" | 4"X14" | 1'–0" |

3 * PIPES DO NOT REQUIRE CURB HIKE

ALHAMBRA FOUNDRY





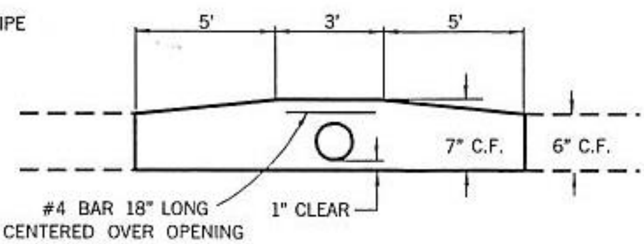




**Figure 3-25: Curb Drain Examples**

### 3.3.2.9 Combination Type Inlets

#### 3.3.2.9.1 General

Combination type inlets are characterized by use of curb and grate inlets acting together. The combination inlets provide use of the features of grate inlets (i.e., no low flow bypass) and curb inlets (i.e., debris pickup before grate inlet). The use of these inlets should be considered by the designer for areas where no bypass flow is desired and where a potential for debris exists. Due to the combined characteristics of the inlets, capacity features of either design are not at a maximum. Grate inlets typically become inoperative during the early portion of the storm hydrograph. This is due to debris such as paper and leaves.

#### 3.3.2.9.2 Typical locations

- Cul-de-sac where driveways reduce inlet dimensions.
- Locations where grate inlets are required and carry-over needs to be minimized.

#### 3.3.2.9.3 Sizing

Two sizes of combination inlets are used in Orange County

- 10-foot length uses a 10’ curb inlet with a 3’-4” grate inlet.
- 7-foot length uses a 7’curb inlet with a 3’-4” grate inlet.

#### 3.3.2.9.4 Design Parameters

Parameters needed to design the combination inlets are:

- Slope of street, $S_0$
- Runoff, Q
- Depth of water at upstream end of inlet, Y

#### 3.3.2.9.5 Capacity

It has been found at lower depths of flow (up to faceplate soffit) that the capacity of the square faceplate is nearly equal to the round faceplate. The designer should be cautious in specifying the square faceplate at depths of flow that exceed hydraulic opening.

Figure 3-26 and Figure 3-27 provide the capacity for the 7 and 10-foot length curved faceplate inlets, respectively. Figure 3-26 and Figure 3-27 may be used for square faceplate inlets if the inlet capacity is reduced by 25%.

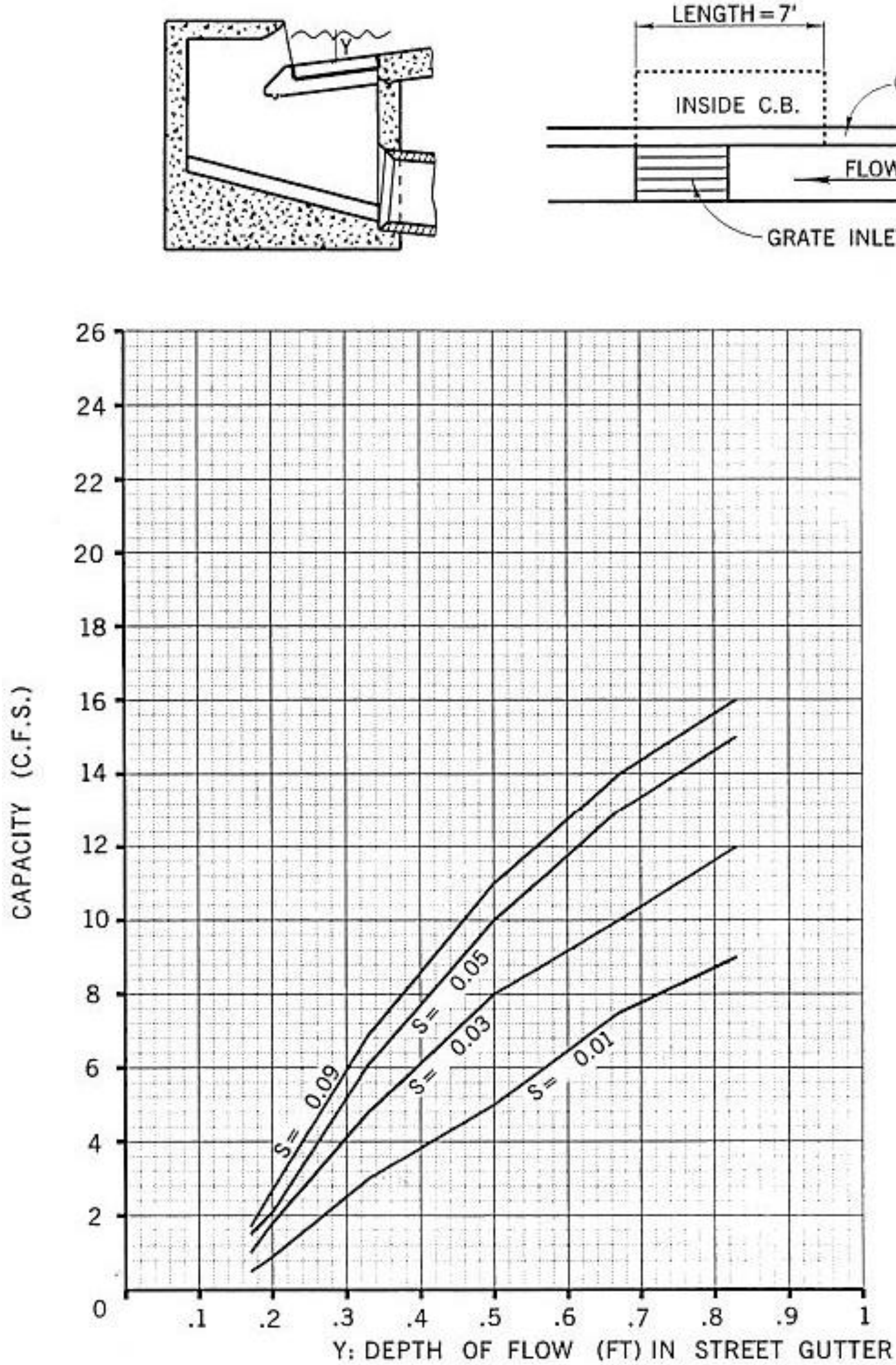



**Figure 3-26: Inlet Type III – Length 7' Capacity Curves, Curved Face Plate**

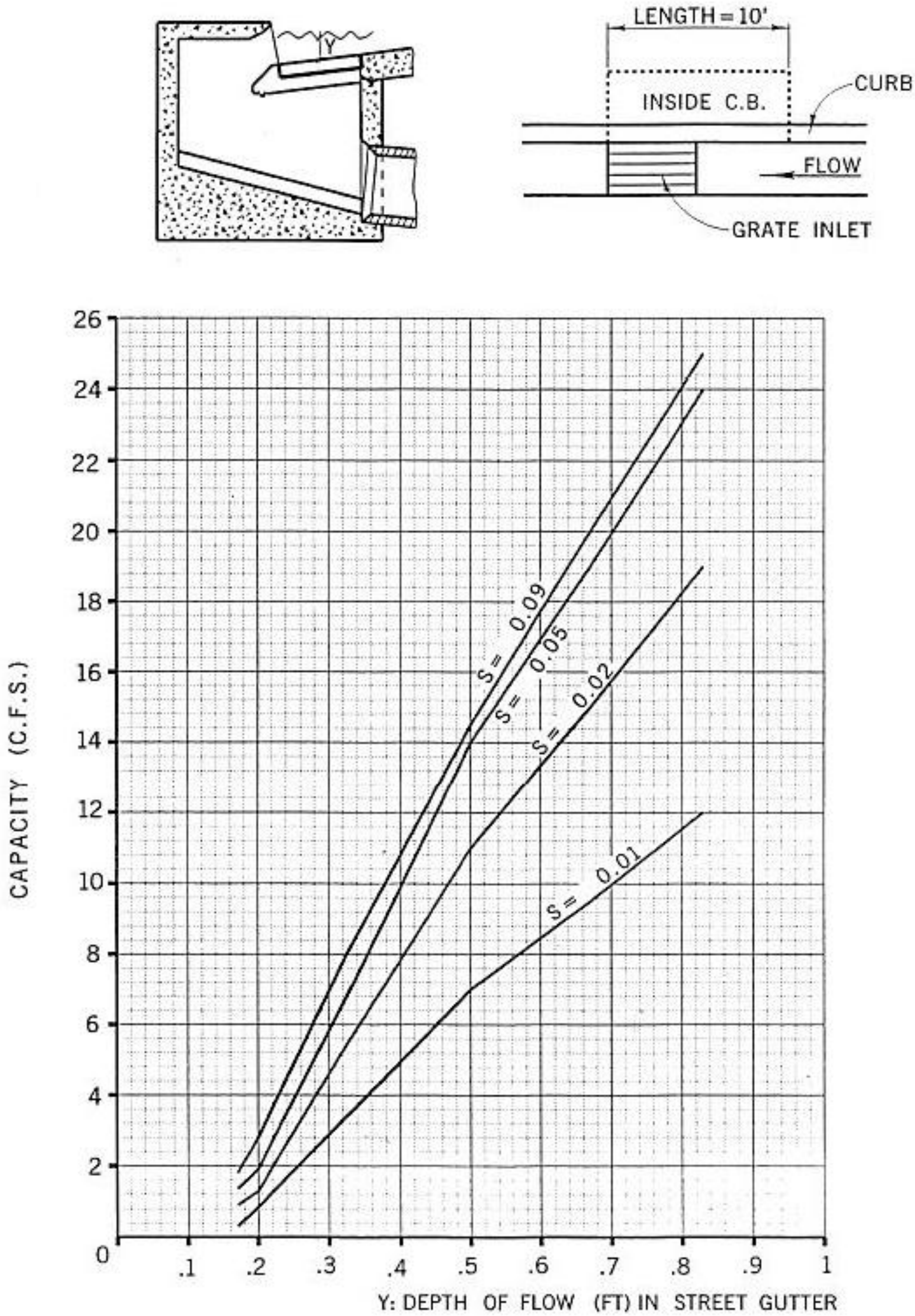



**Figure 3-27: Inlet Type III – Length 10' Capacity Curves, Curved Face Plate**

### 3.3.2.10 Slotted Drain

#### 3.3.2.10.1 General

Wide experience with the debris handling capabilities of slotted inlets suggests restricted use due to clogging. Two common maintenance problems of slotted drain pipes are deposition in the pipe and access for cleaning.

Slotted inlets are effective pavement drainage inlets that have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations.

Flow interception by slotted inlets and curb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement.

Slotted drains will require a hydraulic analysis of the pipe capacity; design shall maintain open flow in the discharge pipe.

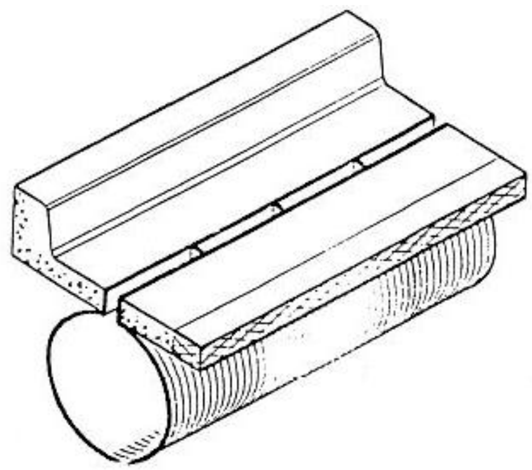


**Figure 3-28: Slotted Drain Inlet**

#### 3.3.2.10.2 Areas of Use/Limitations

Slotted drain pipes may be used in certain approved, select locations or to increase the capacity of curb inlets where justified, providing it meets the following conditions:

- Slotted drain pipe shall not be used in sumps within streets.
- No ADA access nearby
- Use of slotted drain pipe should be discouraged in areas of heavy pedestrian or bicycle traffic. Expanded wire mesh heel guards shall be attached across the top of the open slot when pipe is approved in these areas and is governed by other standards.
- Slotted drain pipes should be used parallel to concrete median barriers for drainage pickup.
- A cleanout or access shall be installed at each end of a run of slotted drain pipe.
- Hot dipped galvanized protection is required for CMP slotted drain pipe.
- Aluminum slotted drain pipe shall not be used.
- Must follow street grade
- Maximum length of any one run of slotted pipe shall be 250'.

#### 3.3.2.10.3 Slotted Drain on Grade Applications

For any given cross slope and longitudinal gutter slope, the required slotted drain inlet length can be determined by the following equation:

$$L_s = [4.762 \times Q^{0.427} \times S^{0.305} \times z^{0.766}]$$

Where:

| | | |
|---|---|---|
| $L_s$ | = | required length of slotted drain for 100% interception (ft) |
| $Q$ | = | flow (cfs) |
| $S$ | = | longitudinal slope (ft/ft) |
| $z$ | = | cross slope reciprocal (ft/ft) |

Example:

For $S = 0.017,\ z = {}^{1}/_{0.017} = 58.82$

It is common practice to carry up to 35% of the discharge/flow to the next inlet. A clogging factor of 50% shall be used.

#### 3.3.2.10.4 Slotted Drain in Sag Locations

Use of slotted drains in sumps within streets is prohibited.

Slotted drains when installed in a sump or low point in grade perform as weirs up to depths of 0.2 feet.

The capacity is defined by the following equation:

$$L_s = \frac{(1.401\ Q)}{Y^{0.5}}$$

Where:

| | | |
|---|---|---|
| $Y$ | = | depth of flow (ft) |
| $L_s$ | = | required length of slotted drain for 100% interception (ft) |
| $Q$ | = | flow (cfs) |

The depth of flow is determined by the street capacity table (see Table 3-4 and Figure 3-29). A clogging factor of 50% shall be used.

At depths greater than about 0.4 ft slotted drains perform as orifices. The interception capacity of a slotted inlet operating as an orifice can be computed by the following equation:

$$Q_i = 0.8\, L\, W\, (2gd)^{0.5}$$

Where:

| | | |
|---|---|---|
| $W$ | = | width of slot (ft) |
| $L$ | = | length of slot (ft) |
| $d$ | = | average depth of water over slot (ft), d ≥ 0.4 ft |
| $g$ | = | 32.17 (ft/s$^2$) |
| $Q_i$ | = | intercepted discharge (cfs) |

For a slot width of 1.75 in the above orifice equation becomes:

$$Q_i = 0.94 L d^{0.5}$$

Where:

| | | |
|---|---|---|
| $L$ | = | length of slot (ft) |
| $d$ | = | average depth of water over slot (ft), d ≥ 0.4 ft |
| $Q_i$ | = | intercepted discharge (cfs) |

Depths between 0.2 ft and 0.4 ft have transitional flow from weir to orifice. The interception capacity of slotted inlets at depths between 0.2 ft and 0.4 ft can be computed by use of the orifice equation. The orifice coefficient varies with depth, slot width, and the length of the slotted inlet.

Figure 3-29 provides solutions for weir and orifice flow for slotted inlet drain capacity. A plot representing data at depths between weir and orifice flow is also provided.

A clogging factor of 50% shall be used.

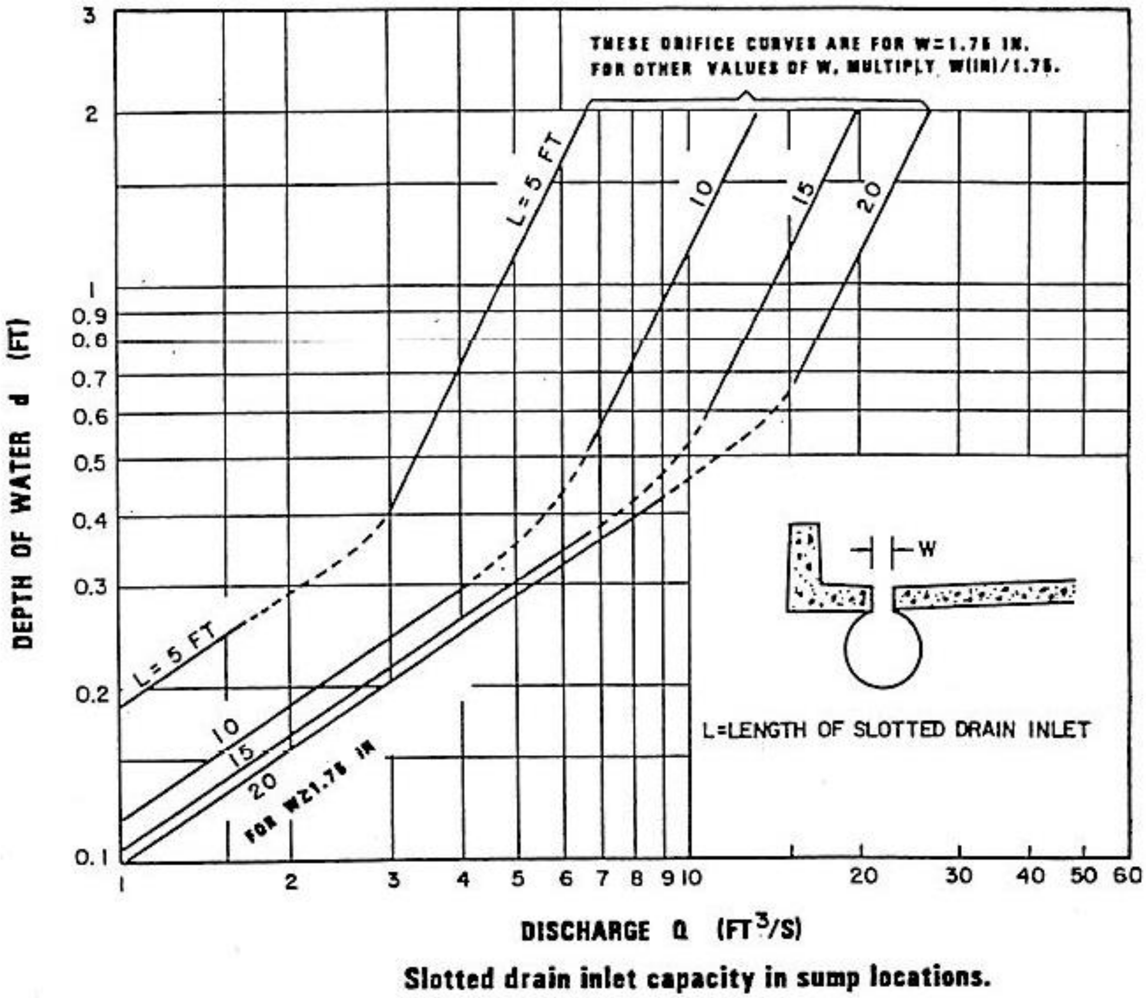



**Figure 3-29: Slotted Drain Capacities**

### 3.3.2.10.5 Slotted Drain in Sheet Flow Applications

In these applications the slotted drain is placed transverse to the direction of flow. The water is not collected and channeled against a berm/curb as required by slotted drain on grade applications.

Capacity may be assumed as 0.04 cfs per foot of length for grades up to 9% and for super-elevation or cross slopes up to 6.25%. A 50% clogging factor shall be used.

#### 3.3.2.10.6 Slotted Drain Example:

**Given:**

Street Slope = 2.9% ($S_0$)

Super-elevation = 4% ($S_e$)

Hydrology Q= 3.8 cfs

Length of slotted drain = 850’ ($L_s$)

**Find:**

Effective length of slotted drain assuming 50% clogged ($LE$)

**Solution:**

$$LE \ = \ 850' \ x \ 0.5 = 425'$$

$$Drain\ Capacity\ (Dc) = \frac{Q}{LE} = \frac{3.8\ cfs}{425\ ft} = \ 0.009\ cfs/ft$$

For slotted drain with:

$$S_0 \leq 9\%$$

and

$$S_e \leq 6.25\%$$

Allowed Capacity is

$$Dc = \ 0.009 \frac{cfs}{ft} < 0.04\ \frac{cfs}{ft}$$

**OK**

#### 3.3.2.10.7 Structural Criteria

- Minimum pipe size shall be eighteen inches (18”).
- Minimum pipe grade shall be one-half percent (0.5%).
- Minimum slot height shall be six inches (6”).
- Pipe shall be 16-gauge minimum.
- Pipe shall conform to the minimum allowable service life for underground conduits.
- All drain pipes/conduits shall be designed to withstand an H-20 loading.
- Maximum length of any one run of slotted pipe shall be 250’.
- The slotted pipe trench shall be backfilled and encased from below the bottom of the pipe with 420 B 2500 concrete to the subgrade of the final surface course of the traveled way.

## 3.3.2.11 Median Type Inlets

### 3.3.2.11.1 General

In landscaped medians, runoff occurring from either rainfall or irrigation should not run onto the street.

### 3.3.2.11.2 Typical Locations

Landscaped medians

### 3.3.2.11.3 Capacity

Median drain shall be sized for 1”/hour of rainfall with flow conveyed to a grate type inlet. The inlet shall be a minimum of 12” x 12” and be based upon Figure 3-23. A 50% clogging factor shall be used. Channel flow shall use a Manning “n” of 0.035.

$$\frac{Q}{P} = 30Y^{1.5}$$

Where:

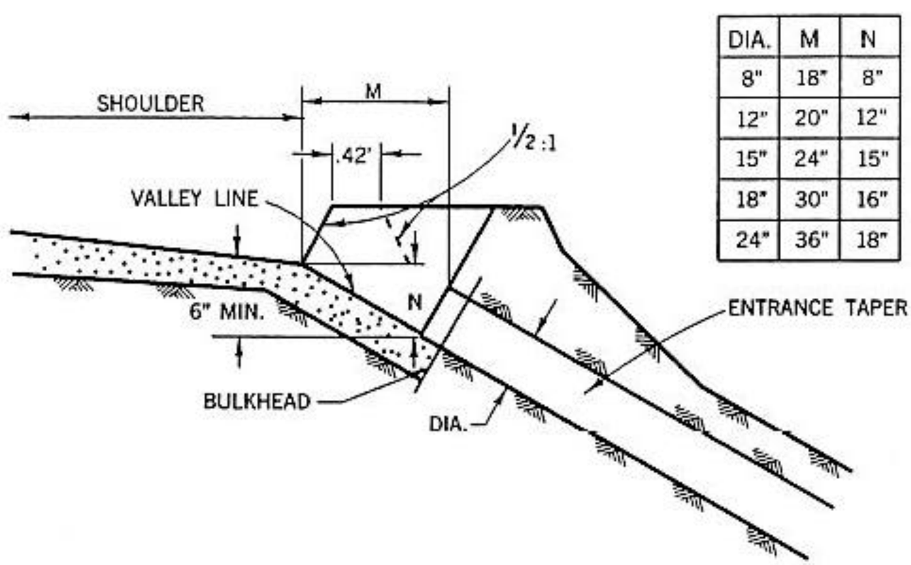
$Y$ = Depth over Inlet (ft)

$P$ = perimeter of drain (ft)

## 3.3.2.12 Over-Shoulder Type Inlets

### 3.3.2.12.1 General

Where concentrated flows of storm water exist on non-curb and gutter roadways, over-shoulder inlets per OCPW Standard Plan 1334 are used to receive the water and discharge it through pipe down drains. This arrangement should be contrasted with curb/gutter inlets that remove concentrated flow from an improved-ultimate roadway surface for collection and conveyance to an outfall.




| DIA. | M | N |
|---|---|---|
| 8" | 18" | 8" |
| 12" | 20" | 12" |
| 15" | 24" | 15" |
| 18" | 30" | 16" |
| 24" | 36" | 18" |

**Figure 3-30: Over-Shoulder Drain**

#### 3.3.2.12.2 Typical Locations

- Where the street is to be widened in the near future
- Rural locations
- Where no curb and gutter exist

#### 3.3.2.12.3 Sizing

Inlet capacity shall be sized using the following formula:

For rectangular sections:

$$Q = 3.086h^{1.5}W_c$$

Where:

| | | |
|---|---|---|
| $Q$ | = | discharge (cfs) |
| $h$ | = | height of opening (ft) |
| $W_c$ | = | width of culvert (ft) |

For circular sections:

$$Q = 2.581\ (D/12)^{2.5}$$

Where:

$Q$ = discharge (cfs)

$D$ = Diameter of drain pipe (in)

The approach velocity longitudinal to the street shall be taken into account when sizing inlets.

Pipe capacity shall be checked using an appropriate backwater curve should outlet control be suspected.

#### 3.3.2.12.4 Drain Pipe

The over side drain pipe shall be verified for hydraulic capacity. The drain pipe may be metal or concrete. If the street is to be widened within 20 years or the roadway is in a rural area and replacement of drain will not obstruct traffic, then a temporary metal pipe may be used.

### 3.3.2.13 Bridge Deck Type Inlets

#### 3.3.2.13.1 General

Bridge deck drains are designed to accept flow from the bridge only. A curb-inlet shall be designed upstream of the bridge to intercept all street flows before the bridge structure. Figure 3-31 shows a diagram of a bridge deck inlet.

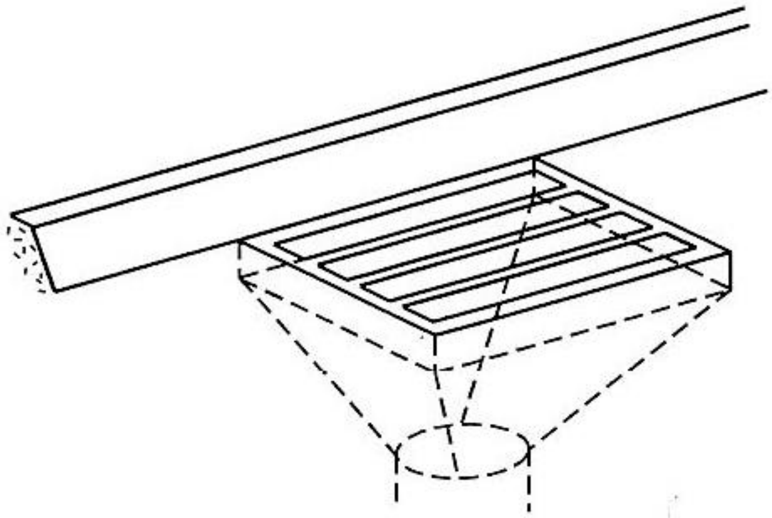


**Figure 3-31: Bridge Deck Inlet**

### 3.3.3 Connector Pipe

#### 3.3.3.1 Calculations of Minimum Inlet Depths and Connector Pipe Sizes

##### 3.3.3.1.1 Single Inlet – Inlet Control

Given the available head (H), the required connector pipe size can be determined from culvert equations such as those given in King & Brater, *Handbook of Hydraulics*, Section Four, fifth edition. Figure 3-32 shows a diagram defining the parameters used to calculate the inlet depth for a single inlet with an inlet control condition. Figure 3-33 can be used for a nomographic solution of a culvert equation for culverts flowing full.

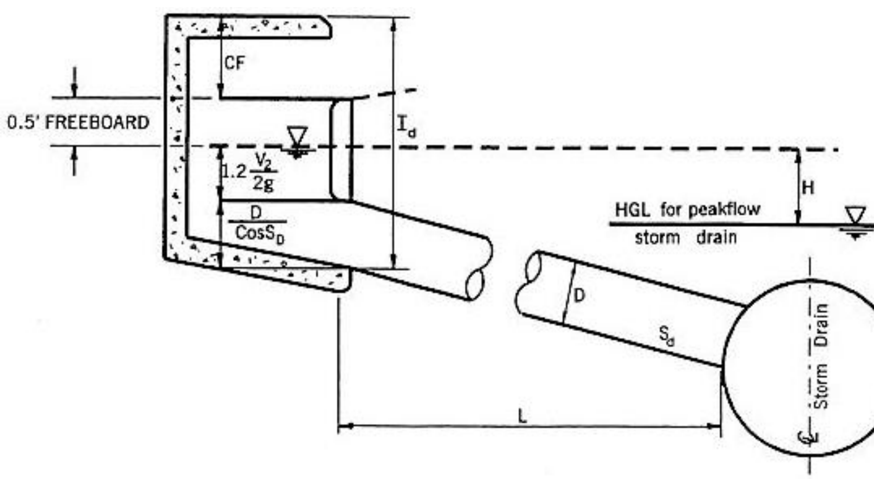



**Figure 3-32: Single Inlet Control – Inlet Diagram**

The minimum catch basin inlet depth "$I_d$" shall be determined as follows:

$$I_d = CF + \text{FB} + 1.2\,\frac{V^2}{2g} + \frac{D}{\cos(S_D)}$$

Where:

| | | |
|---|---|---|
| $I_d$ | = | Depth of the inlet from the invert of the connector pipe to the top of the curb (ft) |
| $CF$ | = | Vertical dimension of the curb face at the inlet opening (ft) |
| $V$ | = | Average velocity of flow in the connector pipe, assuming a full pipe section (ft/s) |
| $D$ | = | Diameter of the connector pipe (ft) |
| $S_D$ | = | Slope of connector pipe (°) |
| $FB$ | = | Freeboard (ft) = 0.5 ft |

Note: The term $1.2\,V^2/2g$ includes an entrance loss of 0.2 of the velocity head.

Assuming a curb face at the inlet opening of 10 inches (6" curb face and 4" local depression) and $\cos(S_D)$= 1 (horizontal inlet pipe), the above equation may be simplified to the following:

$$I_d = 1.33 + 1.2\frac{V^2}{2g} + D$$

Where:

$I_d$ = Depth of the inlet from the invert of the connector pipe to the top of the curb (ft)

$V$ = Average velocity of flow in the connector pipe assuming a full pipe section (ft/s)

$D$ = Diameter of the connector pipe (ft)

Note: The term $1.2\,V^2/2g$ includes an entrance loss of 0.2 ft of the velocity head

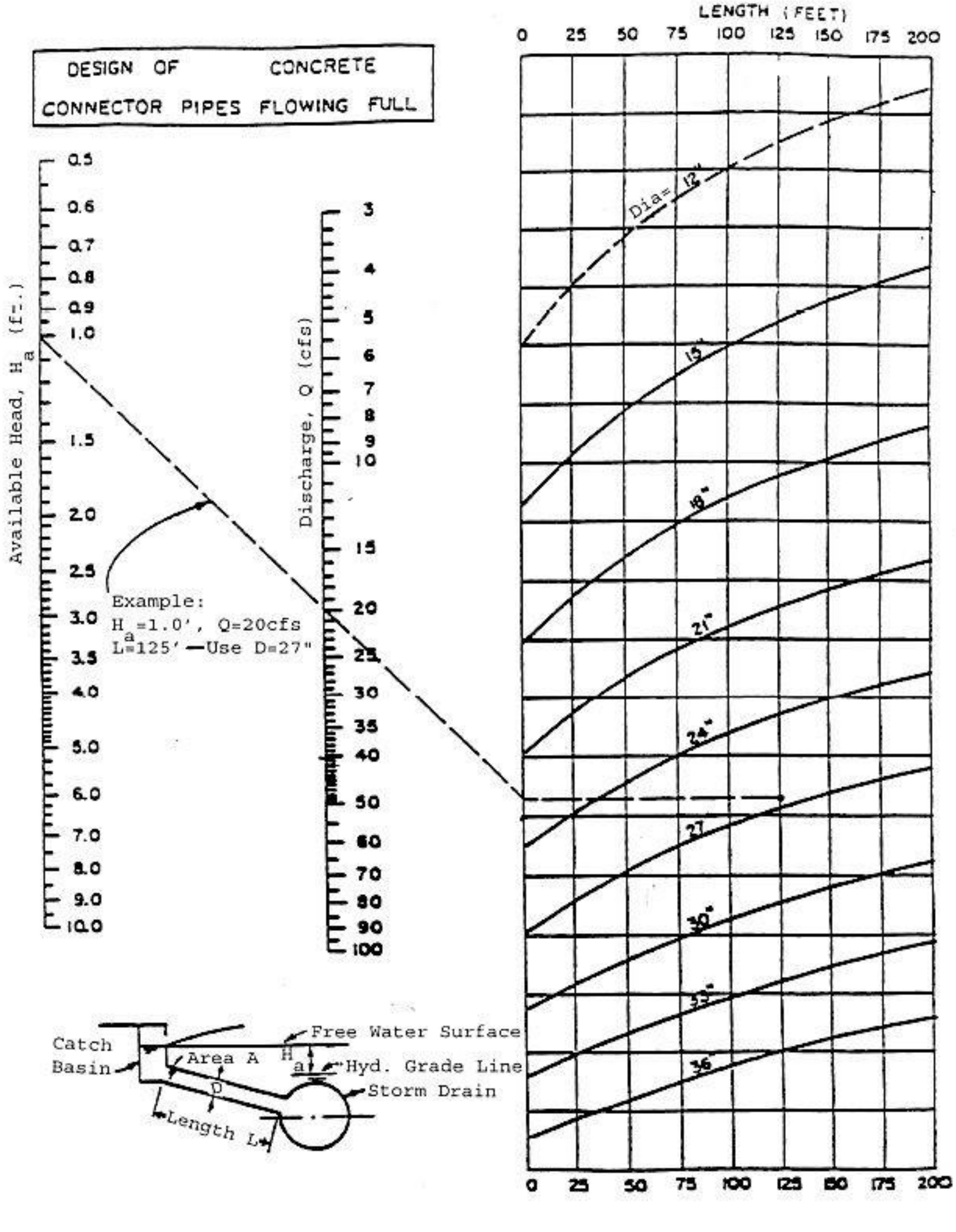



**Figure 3-33: Connector Pipes Flowing Full**

#### 3.3.3.1.2 Inlets in Series – Inlet Control

Select a connector pipe size for each inlet and determine the related head loss ($H_1, H_2$) by means of a culvert equation, or by Figure 3-34. The sum of head losses in the series shall not exceed the available head, i.e., $H_1 + H_2 + \ldots + H_N = H$.

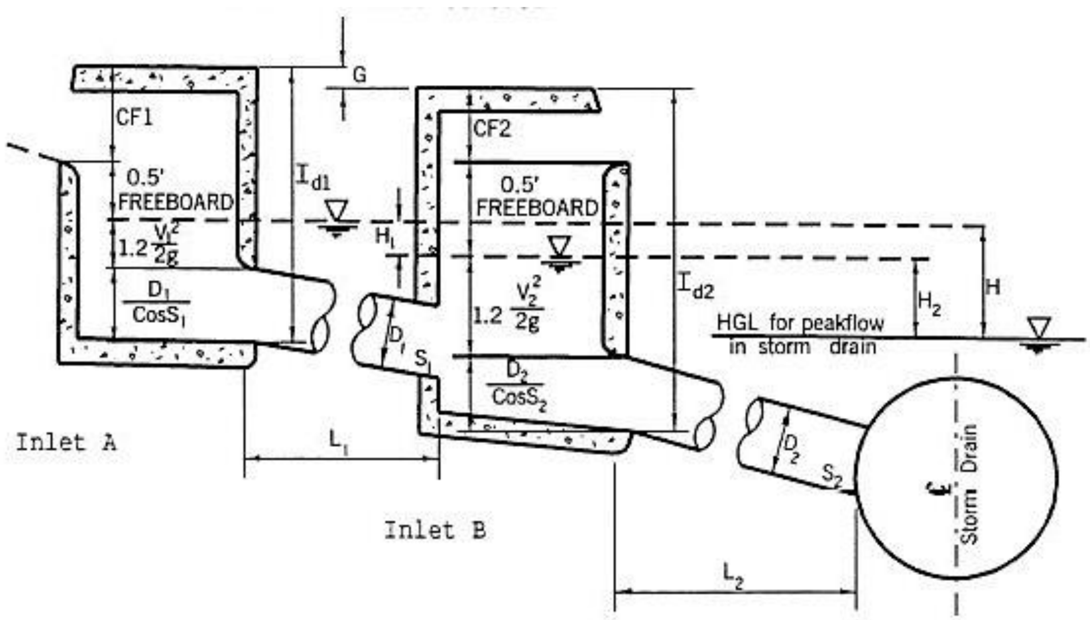



**Figure 3-34: Inlets in Series – Inlet Control Diagram**

The minimum catch basin depths, $I_d$ , shall be determined in the following manner:

$$I_{d_1} = inlet\ depth\ of\ the\ higher\ elevation\ inlet\ (Inlet\ A)$$

$$I_{d_2} = inlet\ depth\ of\ the\ lower\ elevation\ inlet\ (Inlet\ B)$$

The first depth, $I_{d_1}$, shall be calculated as for a single catch basin.

$$I_{d_1} = CF_1 + FB_1 + 1.2\frac{V_2^2}{2g} + \frac{D_1}{\cos(S_1)}$$

Applying the same assumption as the single inlet example:

$$CF_1 = 0.833\ (ft)$$

$$FB_1 = 0.5\ (ft)$$

$$\cos(S_1) = 1$$

Yields

$$I_{d_1} = 1.33 + 1.2\frac{V_1^2}{2g} + D_1$$

The second depth, $I_{d_2}$, shall be determined as follows:

$$I_{d_2} = CF_1 + FB_1 + H_1 + 1.2\frac{V_2^2}{2g} + \frac{D_2}{\cos(S_2)} - G$$

where $G$= difference in elevation between top of curb 1 and top of curb 2

Assuming again that $CF_1 = 0.833\ (ft)$, $FB_1 = 0.5\ ft$, and $\cos(S_2) = 1$

$$I_{d_2} = 1.33 + H_1 + 1.2\frac{V_2^2}{2g} + D_2 - G$$

The freeboard provided for the second inlet shall not be less than 0.5 feet and shall be checked as follows:

$$FB_2 = I_{d_2} - \frac{D_2}{\cos(S_2)} - 1.2\frac{V_2^2}{2g} - CF_2$$

Where:

| | | |
|---|---|---|
| $CF$ | = | height of curb face (ft) |
| $FB$ | = | Freeboard (ft) |

Note:

If $CF_2 = 0.83$ and $\cos\,(S_2) = 1$

Then $FB_2 = I_{d_2} - D_2 - 1.2\frac{{V_2}^2}{2g} - 0.83$

Connector pipes between inlets in series shall be checked for adverse slope by the following relationship:

$$I_{d_2} - 0.5' > I_{d_1} - G$$

The slope shown above is the standard 4:1 cross slope of the inlet floors.

### 3.3.3.2 Hydraulic Gradient Losses Due to Inlets (Pressure Flow)

Figure 3-35 defines the parameters for calculating hydraulic gradient loses in different inlet conditions.

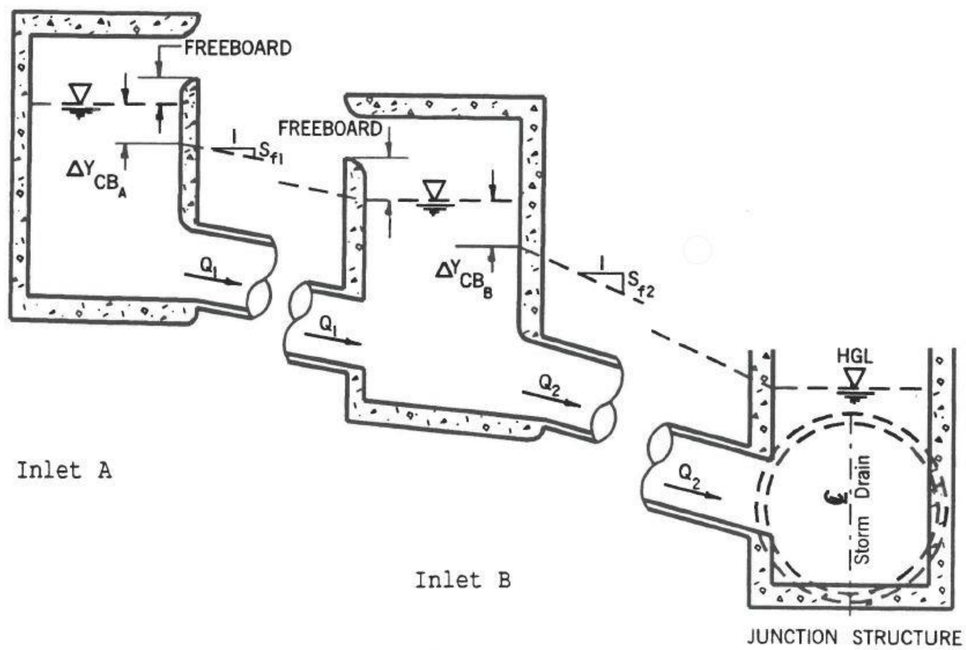



**Figure 3-35: Hydraulic Gradient Losses Due to Inlets**

Inlet A – Inline or in series

$$\Delta Y_{CB_A} = \frac{Q_2 V_2 - Q_1 V_1 cos\theta}{g\frac{A_1 + A_2}{2}} + 0.20\,\frac{V_2^2}{2g}$$

Inlet B – End line or direct lateral from junction structure

$$\Delta Y_{CB_B} = 1.2\,\frac{V_1^2}{2g}$$

The freeboard ($FB$) provided for each inlet generally shall not be less than 0.5 ft.

## 3.4 Acceptable Software

The following are acceptable software applications for Street Flow and Inlet Capture Calculations:

- Bentley Hydraulics
- Federal Highway Administration (FHWA) Hydraulic Toolbox
- Advanced Engineering Software (AES)
- AutoCAD Hydraulic Software

If a computer application other than listed above is used, documentation needs to be provided to OCPW with justification for its use.

## 3.5 References

Brater, E. F., King, H. W., Lindell, J.E., & Wei, C.Y. (1996). *Handbook of Hydraulics (7$^{th}$ edition)*. McGraw-Hill Book Company.

Brown, S.A., Schall, J.D., Morris, J.L, Doherty, C.L., Stein, S.M., & Warner, J.C. (2009). *Urban Drainage Design Manual: Hydraulic Engineering Circular No. 22 (HEC 22), Third Edition*. Federal Highway Administration. FHWA-NHI-10-009, September 2009.

Federal Emergency Management Agency. (1979). The Floodway: A Guide for Community Permit Officials

Federal Highway Administration. (n.d.). *FHWA Hydraulic Toolbox Version 4.2 Desktop Reference Guide*.

King, H. W., Brater, E. F. (1963). *Handbook of Hydraulics (5$^{th}$ edition)*. McGraw-Hill Book Company.

Schall, J. D., Thompson, P. L., Zerges, S. M., Kilgore, R. T., & Morris, J. L. (2012). *Hydraulic Design of Highway Culverts: Third Edition (HDS 5)*. Federal Highway Administration. FHWA-HIF-12-026, April.

# Chapter 4 Storm Drains

## 4.1 Introduction

This chapter discusses the design criteria, hydraulic design, pipe types, inlet design and outlet design for construction of storm drains with the County of Orange. The chapter concludes with discussion of acceptable software for use in storm drain hydraulics modeling and references.

## 4.2 Design Criteria

The following design criteria shall be used for storm drain and local drainage structures built for dedication to the County of Orange, OCFCD, or private facilities within unincorporated Orange County.

Regional or Sub-Regional design storm frequencies are subject to individual review by OCPW and should be in accordance with the Orange County Hydrology Manual, its subsequent addendums, and this manual's Flood Protection Goals. This manual is not intended to supersede any information contained within the Orange County Drainage Area Management Plan (DAMP).

### 4.2.1 Protection Levels

#### 4.2.1.1 Structures

The goal is to provide 100-year protection for all habitable structures pursuant to Public Services and Facilities Element of the County's General Plan. The width and depth shall be a minimum of 1' below a finished floor if FEMA determined base flood elevations are provided on a Flood Insurance Rate Map, or 1' below pad elevation in accordance with Orange County Ordinance No. 09-008 (see Chapter 3).

#### 4.2.1.2 General Criteria

1. Storm drains with tributary areas of less than 640 acres are to be designed for a minimum of 10-year frequency below top of curb using a combination of street and storm drain flow. In sump conditions, catch basins and the connecting storm drains should be designed to a 25-year frequency with secondary overflow or provide 100-year protection.
2. Regional or Sub-Regional design storm frequency is subject to individual review by OCPW and should be in accordance with the Orange County Hydrology Manual, its subsequent addendums, and this manual's Flood Protection Goals. In addition, it must be designed to contain, as a minimum, the Federal Emergency Management Agency's (FEMA's) 100-year discharges used for defining Flood Insurance Rate Map floodplains.
3. The product of the water depth, y (ft), at the curb multiplied by velocity, v (fps), shall not exceed 6 $ft^2/s$ for any street. This criterion applies to storms up to a 25-year frequency.
4. The Rational Method, as described in the Orange County Hydrology Manual and its subsequent addendums, shall be used to compute peak runoff for drainage areas less than 640 acres for sizing local storm drains.
5. Ultimate condition land use should be based upon General Plan Land Use or similar planned communities' surface drainage assumptions shall be used to determine runoff loss rates.
6. Design Engineers are cautioned that hydrology calculations developed before adoption of the current manual for many regional and local drainage facilities may have been based upon rainfall data and land uses less strict than present practice. Use of any hydrology study not

prepared in accordance with the Orange County Hydrology Manual and its subsequent addendums requires prior approval by OCPW.

### 4.2.2 Hydraulic Grade line

A design hydraulic grade line at least 0.5’ below the local depression shall be provided within catch basins for the designed storm drain system. In all other cases, or in preliminary studies that do not consider inlet hydraulics, the design hydraulic grade line of the conveyance facility shall be at least 2 feet below street gutter grade and the design hydraulic grade line in the main trunk shall be sufficiently below surrounding ground to accommodate local drainage. Preliminary design in undeveloped areas shall assume that future street elevations will be 2 feet below average existing ground.

#### 4.2.2.1 Hydraulics

1. A storm drain profile that includes the hydraulic grade line and table of appropriate hydraulic design data ranges shall be provided on final file plans. Station, section, bed slope, Manning’s “n”, design flow, velocities, and frequency must be included in the plans. Normal depth, critical depth, and Froude number shall also be shown for open channel flow.
2. Storm flows picked up in a storm drain system shall remain in the system until discharged into an acceptable point of disposal.
3. Branching of flow or parallel storm drain systems are not permitted in new facilities.
4. Where calculated backwater at a street crossing raises upstream water surface more than 1’ in a natural channel, a floodplain easement shall be provided in the design. The 100-year level of protection for habitable structures shall not be replaced or superseded by these criteria.

#### 4.2.2.2 Minimum Permissible Velocities for Underground Systems

Covered or underground systems shall be self-cleaning (i.e., velocity in pipe is adequate to clean the storm drain and not less than 3 fps for the design flowrate of the pipe). The Design Engineer shall verify various flow characteristics (i.e., sediment, debris) confirming that underground system is self-cleaning at low flow when historical data is not available.

#### 4.2.2.3 Maximum Permissible Velocities

Maximum permissible velocities vary by storm drain type and can be verified by Caltrans Highway Design Manual Table 855.2A.

### 4.2.3 Manning's Roughness Coefficient

| Material/Conveyance Type | "n" Value |
|---|---|
| Plastic Pipe (Smooth) | 0.012 |
| CMP Full Asphalt Spun Lined | 0.013 |
| CMP Not Lined | |
| 2-2/3" x 1/2" Corrugations | 0.025 |
| 3" x 1" Corrugations | 0.028 |
| 6" x 2" Structural Plate | 0.035 |
| 9" x 2-1/2" Structural Plate | 0.035 |
| RC Pipe Spun | 0.013 |
| RC Pipe Dry cast | 0.013 |
| PCC Box & Arch Sections Trowel Finish | 0.013 |
| PCC Cast-in-Place Pipe | 0.014 |
| SRP | 0.015 |
| PCC Trapezoidal Channel | 0.015 |
| Vertical Wall Channel | 0.014 |
| Flood Plains | |
| Pasture or Cultivated | 0.040 |
| Heavy weeds, light brush | 0.050 |
| Medium to dense brush | 0.090 |
| Willows | 0.170 |

**Table 4-1: Manning's Roughness Coefficient for Storm Drains (Caltrans, 2018 and County of Orange, 2000)**

### 4.2.4 Alignment and Curvature

#### 4.2.4.1 Storm Drain Alignment

1. Storm drain laid within the roadway shall be placed at least 30" below the roadway surface. (County Ordinance 1961§ 6-3-69)
2. Storm drain cover shall not be in excess of 15' (measured from top of pipe to finish ground), without OCPW's approval.
3. Work within the existing County of Orange or OCFCD R/W will require a permit from the County Property Permits.
4. Storm drain laid parallel to the Arterial Highway shall be placed so the manhole centerline is within the parking lane or center of the outside travel lane. Storm drain shall not be placed under curb and gutter. Placement under sidewalk is not recommended.
5. The minimum slope for a storm drain main line shall be 0.001 (.10 %), unless otherwise approved by OCPW.
6. Storm drain shall be located such that their installation or removal can be accomplished by an open cut without interfering with slope buttress, retaining walls, or toe of slopes.
7. Alternative storm drain designs shall have separate hydraulic and structural calculations and note design parameters on the plans.
8. Upon award of contract and before placement of pipe, the Design Engineer shall revise the Plans to indicate the actual design used.

## 4.2.5 Transition from Large to Small Storm Drain

### 4.2.5.1 General

Storm drains shall be increasing in size in the downstream direction. However, when studies indicate it may be advisable to decrease the size of a downstream section, the storm drain may be decreased in size in accordance with the following limitations:

1. For slopes of less than 0.0025 (0.25 %) and when storm drain size is less than 48” in diameter, reduction will not be allowed.
2. For continuous slopes of more than .0025, storm drain sizes of 48” in diameter or greater may be reduced upon OCPW’s approval. Each reduction is limited to a maximum of 3” for pipe 48” in diameter or smaller and to a maximum of 6” for pipe larger than 48” in diameter. Multiple reductions shall be separated by a minimum of 4’. Inverts should be matched across structures.
3. Where storm drains are to be reduced in size due to a change in grade, decreases in pipe size shall be based upon upstream size and be limited to 3” for sizes less than and including 48” and 6” for sizes larger than 48”. Inverts should be matched across structures.
4. Transitions required by reductions will require additional clean-out manholes and require OCPW review and approval.

Prior discussion with OCPW is recommended for these designs.

### 4.2.5.2 Downstream Size Reduction

Where storm drains are to be reduced in size due to a change in grade, the criteria for locating the transition shall be as shown below:

$$S_o L + d_1 + \frac{V_1^2}{2g} = d_2 + \frac{V_2^2}{2g} + 0.1\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) + S_f L + h_m$$

And

$$L = \frac{d_2 - d_1 + 1.1\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) + h_m}{S_o - S_f}$$

Where:

| | | |
|---|---|---|
| $L$ | = | minimum length from grade break to pipe reduction (ft) |
| $d_1$ | = | diameter or depth of larger storm drain (ft) |
| $d_2$ | = | diameter or depth of smaller storm drain (ft) |
| $V_1$ | = | velocity in larger storm drain flowing full (ft/s) |
| $V_2$ | = | velocity in smaller storm drain flowing full (ft/s) |
| $S_o$ | = | slope (ft /ft) |
| $S_f$ | = | friction slope (ft /ft) |
| $h_m$ | = | other losses occurring between the transition and the grade break such as bend and confluence losses (ft) |

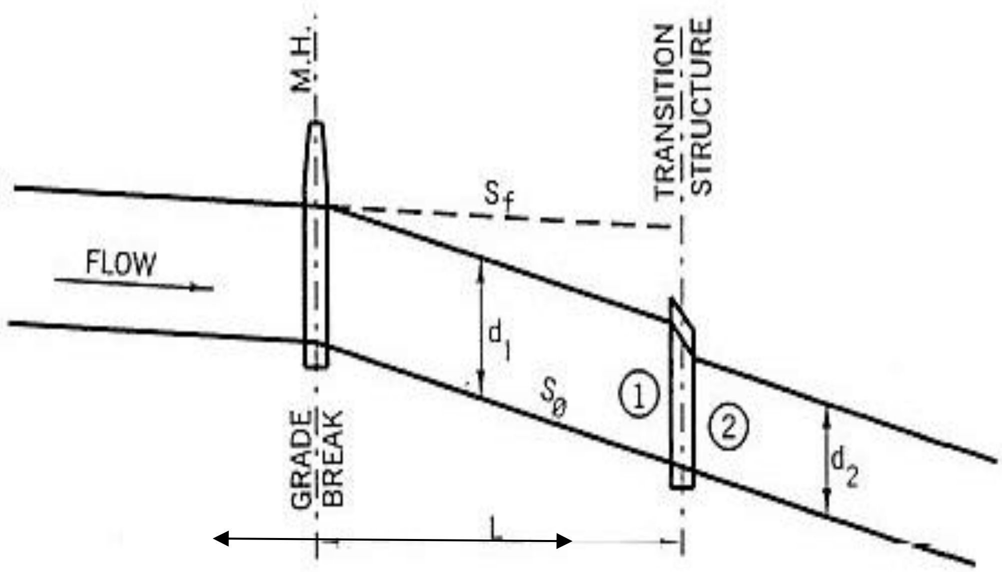



**Figure 4-1: Downstream Pipe Reduction**

#### 4.2.5.3 Example Problem

$Q = 400\ cfs$

$d_1 = 84" = 7\ ft \qquad d_2 = 78" = 6.5\ ft$

$A_1 = 38.49\ ft^2 \qquad A_2 = 33.18\ ft^2$

$V_1 = 10.4\ fps \qquad V_2 = 12.0\ fps$

$\frac{V_1^2}{2g} = 1.68\ ft \qquad \frac{V_2^2}{2g} = 2.24\ ft$

$S_0 = 0.00474$

$S_f = 0.00395$

$$L = \frac{6.5 - 7.0 + 1.1(2.24 - 1.68)}{0.00474 - 0.00395} = 147\ ft$$

#### 4.2.5.4 Branching of Flow in Pipe

Branching of flow in pipe or parallel storm drain systems are not permitted in new facilities.

### 4.2.6 Required Maintenance Easement Widths

The width of public storm drain easements shall be determined as follows:

1. The distance from spring line of the storm drain to ground level multiplied by two (sides) plus the outside width of storm drain trench (see Figure 4-2). Where easements extend into slopes or cuts, the easement shall be extended to the daylight line.

2. Where inside maintenance is required as specified in "Storm drains with high covers" (below), easement widths shall be the outside width of the storm drain plus 2 feet on each side.
3. Minimum width of an easement shall be 10'.
4. No structural encroachments shall be allowed within easements.
5. Easement widths criteria shall apply to pipe sizes less than 60". For pipe size equal to or greater than 60", easement widths shall be determined on a case-by-case basis.
6. These easement criteria shall not apply to existing or future OCFCD facilities.

### 4.2.7 Deep-Cover Storm Drains and Culverts

Storm drains with high covers (excepting connector pipe lengths less than 200'):

1. If cover to top of storm drain is less than 20 ft, standard criteria for storm drain design shall be followed.
2. If cover to top of storm drain is equal to or greater than 20 ft:
    a. When cover exceeds 20', it is assumed maintenance will be accomplished from within the pipe; therefore, the minimum size shall be 60" in diameter/height. The culvert shall be oversized 12" (in height and width) above hydraulic needs to provide for future interior repairs and shall be reviewed and approved by OCPW before final design. The oversize pipe shall extend to acceptable maintenance access points as approved by OCPW.
    b. Depth of storm drains over 40' requires a special design to be approved by the Chief Engineer of OCPW.
3. For reaches downstream of high fills, storm drain shall be downsized in accordance with Section 4.2.5.

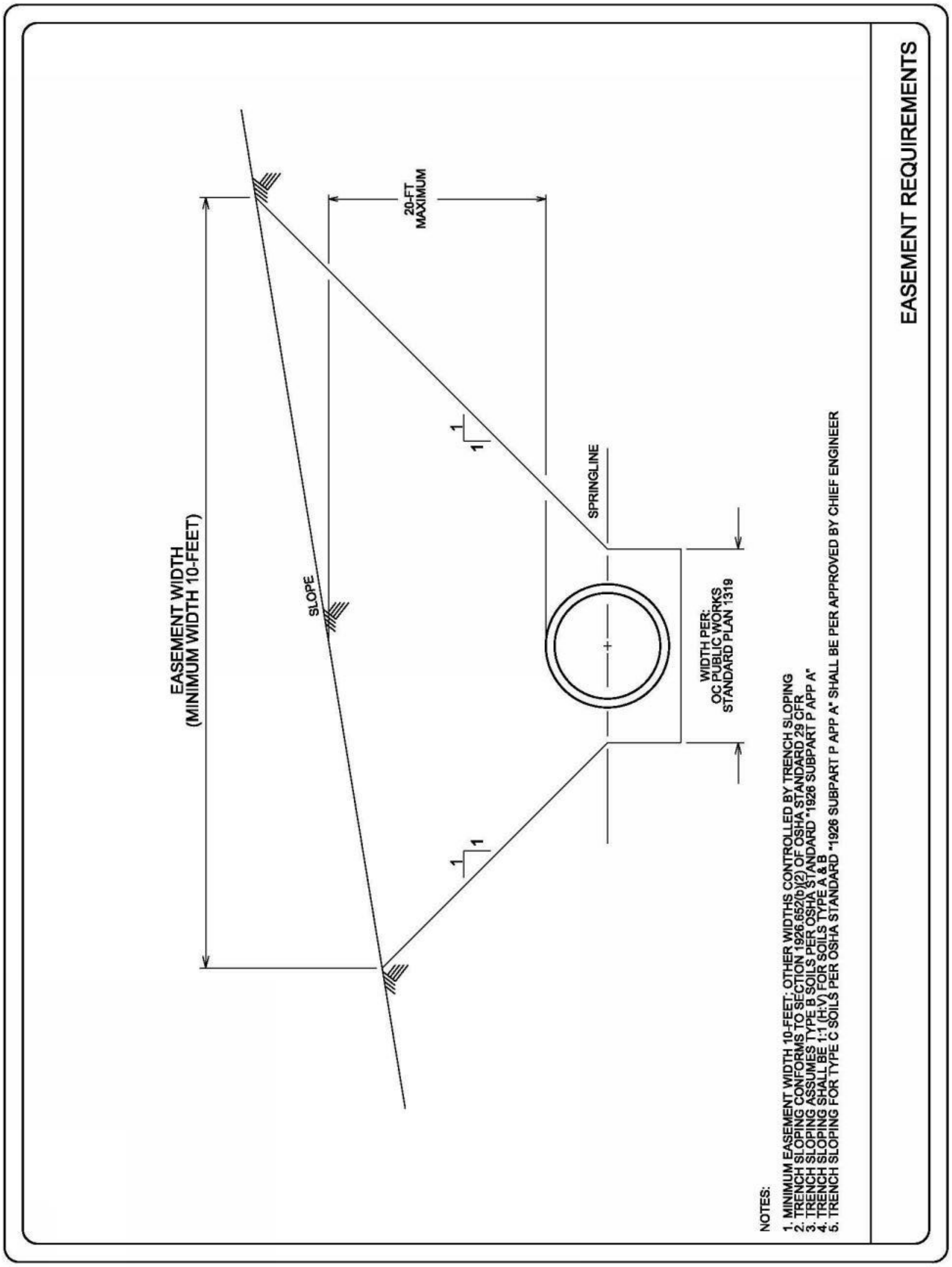



**Figure 4-2: Easement Requirements**

## 4.2.8 Storm Drain Maintenance and Access Criteria

### 4.2.8.1 Manholes

#### 4.2.8.1.1 Spacing

Storm drain diameter 30” or smaller: Manholes shall be spaced at intervals of approximately 300’. Where the proposed storm drain is less than 30” in diameter and the horizontal alignment has bends or angle points, the manhole spacing shall be reduced to approximately 200’.

Storm drain diameter larger than 30” but smaller than 45”: Manholes shall be spaced at intervals of approximately 400’. When angle points occur in excess of 15 degrees, additional manholes shall be provided.

Storm drain diameter 45” or larger: Manholes shall be spaced at intervals of approximately 500’. When angle points occur in excess of 15 degrees, additional manholes shall be provided.

#### 4.2.8.1.2 Location

1. Manholes shall not be located in street intersections.
2. In situations where the proposed storm drain is to be aligned both in easement and in street R/W, manholes shall be located in street R/W, wherever possible.
3. Manholes shall be located as close to changes in grade as feasible when the following conditions exist:
    a. When the upstream storm drain has a steeper slope than the downstream storm drain and the change in grade is greater than 10 percent, sediment tends to deposit at the point where the change in grade occurs.
    b. When transitioning to a smaller downstream storm drain due to an abruptly steeper slope downstream, debris tends to accumulate at the point of transition.
4. Where storm drains cross onto private property, manholes shall be located at the end of the storm drain on public property.

### 4.2.8.2 Pressure Manholes

A pressure manhole shaft and a pressure frame and cover shall be installed whenever the design hydraulic grade line is above the ground surface.

### 4.2.8.3 Deep Manholes

A manhole shaft safety landing shall be provided when vertical access is 20’ or greater in depth. Also, a 36” manhole frame and cover shall be used. See OCPW Standard Plan 326-OC & 1508 (Revised April 2018).

### 4.2.8.4 Cleanouts

Cleanouts are structures that allow access for maintenance of a storm drain facility for use in private systems. The design engineer shall specify cleanouts at prescribed locations within a storm drain facility, and at specific locations in relation to the horizontal and vertical curvature of a pipe alignment.

#### 4.2.8.4.1 General Location

Cleanouts shall be located at prescribed locations within a storm drain alignment to provide a maintainable drainage system:

1. At the point where a storm drain facility transfers from private to public maintenance, or enters or exits a public R/W; and
2. At points where the storm drain pipe size changes.

#### 4.2.8.4.2 Horizontal Curves and Angle Points

Cleanouts shall also be located within road alignments with horizontal curves as follows:

1. Within 50’ of the end of all horizontal curves;
2. At the point of compound curvature (PCC) and point of reverse curvature (PRC) of all curves; and
3. All horizontal angle points, except as described below.
   a. A single horizontal angle point of 10 degrees or less is allowed without a cleanout in cases where:
      - The angle point does not connect a horizontal and vertical curve; and
      - The angle point is located within 50’ of another cleanout or outfall.

#### 4.2.8.4.3 Cleanouts: Circular Curves in a Vertical Plane or/and Angle Points

Cleanouts shall be located within road alignments with circular curves within the vertical plane as follows:

1. The end of a circular curve within the vertical plane as it intersects a pipe with flatter grade, unless that intersection is within 50’ of another structure;
2. All vertical angle points, *except* as noted in (3), below; and
3. A single vertical angle point of 10 degrees or less is allowed without a cleanout in cases where:
   a. The angle point does not connect a horizontal and vertical curve; and
   b. The angle point is located within 10’ of another cleanout or outfall.

Manufactured, watertight deflection points may be used to obtain up to 30 degrees of vertical deflection within 10’ of a storm drain outfall. When the storm drain is less than 48” in diameter and the outfall is extended downgrade during future construction, this deflection point shall be replaced with a clean-out structure.

### 4.2.8.5 Inlets into Main Line Drains

Lateral storm drain entering a main storm drain shall be connected in accordance with OCPW Standard Plans.

### 4.2.8.6 Minimum Pipe Size

The minimum diameter of publicly maintained storm drain shall be 18”, privately maintained storm drain in public R/W shall be a minimum diameter of 18”.

### 4.2.9 Water-Tight Joints

Watertight joints shall be specified in the following conditions:

1. Where the Hydraulic Grade Line (HGL) will exceed the inside crown (soffit) of the pipe by more than 5’ for more than 40’ of pipe length for the design storm.
2. Where pipe grade exceeds 20 percent.
3. Where the pre-project geologic investigation (i.e., soils report) indicates that current or historic groundwater levels might exceed the pipe invert elevation.
4. For rubber gasket pipe, 10 feet on either side of cut/fill lines.

Special considerations are required where contaminated groundwater is known to exist.

### 4.2.10 Slope Drains and Anchors

Slope anchors will be installed for slope drain pipe with slopes greater than 7-feet vertical interval on all pipe slopes of 5:1 (20%) or steeper. See OCPW Standard Plan 1333. Refer to SPPWC 221-2.

#### 4.2.10.1 Areas of Use/Limitations

Slope drain may be permanent installations or temporary drains for a future extension of a permanent installation, above or below ground.

Any pipe slope drain that would be conspicuous or placed in landscaped areas shall be concealed by burial or other means. For slope drainage, see Grading Code. All above ground pipes are considered design deviations and will require review and approval by OCPW.

#### 4.2.10.2 Structural Criteria

Structural Criteria for Slope Drains shall comply with the following:

1. All slope drains shall have positive watertight joints.
2. Slope drain pipe shall conform to the minimum allowable service life for underground storm drain.
3. Adequate anchorage shall be installed at 7’ vertical intervals for all storm drain pipe placed on or within slopes 5:1 or steeper.
4. Cutoff walls shall be installed at intervals up to a maximum of 30’ horizontally for all pipes placed in slopes where there is the possibility of erosion of the pipe trench on the slope.
5. Where required, cut-off walls shall be reinforced masonry or reinforced cast-in-place concrete.

### 4.2.11 Bulking Factor

In areas where natural open space drains to a proposed storm drain, a bulking factor (increased design Q) may be used in lieu of a debris facility. For discussions on debris facilities and Bulking Factor calculations, see Chapter 9.

### 4.2.12 Pipe Abrasion

In cases where a storm drain pipe is expected to carry a large amount of debris or abrasive sediment material, it shall have measures to provide sufficient design life for the facility. The pipe material will dictate the type and degree of protection required. When protection is warranted, the invert of the pipe (i.e., the bottom 90 degrees of the pipe) shall be protected on all straightaways, and the invert and walls (i.e., the lower 180 degrees of the pipe) shall be protected on all curves. See Caltrans HDM Table 855.2A for abrasion criteria for various pipe types.

## 4.2.13 Abandonment of Facilities

### 4.2.13.1 General

Abandonment of storm drains or other structures that create an unmaintained void in public R/W shall not be permitted. All abandoned voids shall be removed or backfilled with an appropriate structural material to eliminate the possibility of settlement or collapse. Crush in-place abandonment methods shall not be used in public R/W.

### 4.2.13.2 Storm Drains

Storm drains, where feasible shall be removed. Where traffic or structural requirements prohibit removal, one-sack cement slurry shall be used to fill the voids. The storm drain shall have a double brick and mortar plug provided at each end of the line.

### 4.2.13.3 Utility Lines/Conduits

Utility conduits to be abandoned shall be removed in new construction. Where abandoned in existing street, conduits with over 4' of cover may remain, if the inside diameter does not exceed 3". Larger diameters shall receive a one-sack slurry fill.

### 4.2.13.4 Manholes/Vaults

Manholes or vaults shall be removed a minimum of 36" below the top of roadway surface and a concrete lid placed over the manhole/vault after filling with one sack slurry.

### 4.2.13.5 Inlets

Abandoned inlets shall be removed and disposed of at an appropriate disposal site. When a storm drain is to be abandoned within specified limits, all inlets within these limits shall also be removed.

### 4.2.13.6 Plan of Abandonment

A plan for abandonment shall be provided with new construction and retained for future reference. In the case of a permit entry, a copy of this plan shall be attached to the permit.

## 4.2.14 Allowable Materials

Materials to be used for construction of a public drainage system or in public R/W include reinforced concrete pipe, and high-density polyethylene/polypropylene under specific conditions. High-density polyethylene/polypropylene can only be used with prior approval by OCPW, and the Design Engineer must justify its use over RCP. The specified material shall conform to Section 207 of the Standard Specifications for Public Works Construction and have a minimum design life of 100 years. Other pipe materials to be used in public R/W require approval of the Chief Engineer and shall demonstrate a 100-year design life.

A wide variety of materials may be used for construction of a private drainage system outside of public R/W. These materials may include reinforced concrete pipe, corrugated steel pipe, corrugated aluminum pipe, high-density polyethylene, and other materials. The specified material shall be approved by OCPW and shall conform to Section 207 of the Standard Specifications for Public Works Construction. Any system proposed to be dedicated to the County shall be limited to the materials acceptable for work within the public R/W.

The selection of pipe material shall consider factors such as strength of the storm drain under maximum or minimum cover, bedding and backfill conditions, anticipated loading, length of sections, ease of installation, and corrosive action of surrounding soils, expected deflection, and cost of maintenance. Where field conditions indicate the use of one pipe material in preference to others (for instance, corrosive soil conditions, presence of a groundwater table, or a seawater outfall), the reasons shall be clearly presented in the plans and specifications.

### 4.2.15 Storm Drain Plans

Storm drain plans should include plan and profile drawings on 22” x 34” or 24” x 36” sheets. An example is shown in Figure 2-3: Example Typical Plan and Profile Sheet.

## 4.3 Hydraulic Design of Storm Drains

### 4.3.1 Basic Design Procedure

Most procedures for calculating energy grade-line profiles are based on the Bernoulli equation. Terms in the Bernoulli equation are shown in Figure 4-3. This equation can be expressed as follows:

$$\frac{{V_1}^2}{2g} + D_1 + S_{\o}L = \frac{{V_2}^2}{2g} + D_2 + S_f\ L + h_{minor}$$

Where:

| | | |
|---|---|---|
| $D_1$ | = | Vertical distance from invert to HGL at Location 1 (ft) |
| $D_2$ | = | Vertical distance from invert to HGL at Location 2 (ft) |
| $S_{\o}$ | = | Invert slope (ft/ft) |
| $L$ | = | Length of storm drain (ft) |
| $S_f$ | = | Average friction slope between sections 1 and 2 (ft/ft) |
| $V_1$ | = | Average Velocity ($Q/A$) at Location 1 (ft/s) |
| $V_2$ | = | Average Velocity ($Q/A$) at Location 2 (ft/s) |
| $g$ | = | gravity at 32.2 (ft/s$^2$) |
| $h_{minor}$ | = | Minor head losses (ft) |

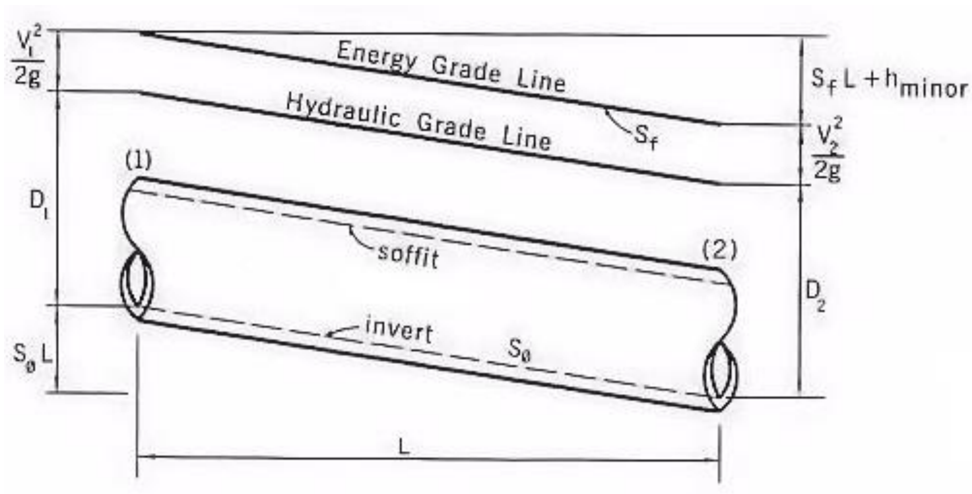



**Figure 4-3: Mainline Storm Drain Definition Sketch**

Minor losses have been included in the Bernoulli equation because of their importance in calculating hydraulic grade-line profiles.

When specific energy ($E$) is substituted for the quantity $\frac{V^2}{2g} + D$ in the above equation and the result rearranged, the following equation is created:

$$L = \frac{E_2 - E_1}{S_{\emptyset} - S_f}$$

Where:

| | | |
|---|---|---|
| $L$ | = | Length of storm drain (ft) |
| $E_1$ | = | Specific Energy at Location 1 |
| $E_2$ | = | Specific Energy at Location 2 |
| $S_{\emptyset}$ | = | Invert slope (ft/ft) |
| $S_f$ | = | Average friction slope between sections 1 and 2 (ft/ft) |

This equation, although a simplification of the complex equation, allows a convenient method for locating the approximate point where pressure flow may become unsealed. The Direct Step Equation will not work for pressure flow—only gradually varied flow.

The Froude number in open-channels shall be maintained less than 0.9 or greater than 1.2. Exceptions must receive prior approval from OCPW before final plans are prepared.

Hydraulics shall be based on design size; storm drains that are oversized due to deep cover shall not be used for hydraulics (this oversized pipe allows new pipe or repairs to be made without reducing hydraulic capacity). Hydraulic design of connector pipes (laterals) between catch basins and storm drain lines are discussed in Section 3.3.3 Connector Pipe.

### 4.3.2 Storm Drain Analysis – Uniform Flow

When a storm drain is not flowing full, the storm drain operates as an open channel and the hydraulic properties can be calculated using open channel techniques. The flow in a storm drain operating as an open channel can be evaluated numerically using the Uniform Flow Equation.

$$Q = \frac{1.486}{n}\, A\, R^{2/3}\, S_f^{1/2}$$

Where:

| | | |
|---|---|---|
| $Q$ | = | flow rate (cfs) |
| $n$ | = | Manning roughness coefficient (no dimension) |
| $A$ | = | flow area ($ft^2$) |
| $R$ | = | hydraulic radius (ft) |
| $S_f$ | = | friction slope (ft/ft), assumed equivalent to storm drain's longitudinal slope, ($S_ø$) |

During full-flow conditions, the flow area and hydraulic radius for a circular pipe of diameter (D) can be simplified to the following relationships:

$$A = A_{full} = \frac{\pi D^2}{4}$$

Where:

| | | |
|---|---|---|
| $A$ | = | flow area ($ft^2$) |
| $D$ | = | diameter of pipe (ft) |

**Equation for Hydraulic Radius for Full Flow Conditions**

$$R = R_{full} = \frac{D}{4}$$

Where:

| | | |
|---|---|---|
| $R$ | = | hydraulic radius (ft) |
| $D$ | = | diameter of pipe (ft) |

Therefore, the minimum required diameter for a circular pipe ($D_r$) needed to convey a particular design flow ($Q$) can be calculated as:

$$D_r = \left(\frac{2.16nQ}{\sqrt{S_ø}}\right)^{3/8}$$

Where:

| | | |
|---|---|---|
| $D_r$ | = | minimum required diameter for a circular pipe (ft) |
| $Q$ | = | flow rate (cfs) |
| $n$ | = | Manning roughness coefficient (no dimension) |
| $A$ | = | flow area ($ft^2$) |
| $R$ | = | hydraulic radius (ft) |
| $S_f$ | = | friction slope (ft/ft), assumed equivalent to storm drain's longitudinal slope, ($S_ø$) |

The pipe diameter is specified as the next standard pipe size larger than the minimum required ($D_r$). An analogous procedure can be followed for alternative storm drain shapes.

### 4.3.3 Storm Drain Analysis – HGL Calculations

The designer shall check the available energy at all junctions and transitions to determine whether the flow in the storm drain will be pressurized due to backwater effects, even when the design flow is less than the full flow capacity of the storm drain. When a storm drain is flowing under a pressure flow condition, the friction slope ($S_f$) and longitudinal slope of the storm drain ($S_o$) may not be equivalent.

To calculate the Energy Grade Line (EGL) for a storm drain system, divide the system into "runs" of pipe between structures (clean-outs, inlets, junctions, or other structures) or changes in grade. The slope of the pipe shall be constant within each run. Starting with the downstream control elevation ($EGL_i$) for the most downstream run of pipe, first calculate the friction losses and bend losses through the pipe and then the losses across the upstream drainage structure. The EGL at the upstream end of the run ($EGL_{i+1}$) will be the sum of the downstream control elevation, friction losses, and structure losses, and will be the downstream control elevation for the next run of pipe:

$$EGL_{i+1} = EGL_i + (\sum H_L)_{PIPE} + (\sum H_L)_{STRUCTURES}$$

Figure 4-3 illustrates the components used in the energy grade line and head loss calculations. The hydraulic grade line (HGL) is then calculated by subtracting the velocity head ($v^2/2g$) from the energy grade line:

$$HGL_i = EGL_i - \frac{v_i^2}{2g}$$

EGL elevations must always decrease in the downstream direction and must always increase in the upstream direction. On the other hand, HGL elevations may increase or decrease at structure locations regardless of the direction considered. For instance, the HGL will increase in the downstream direction within a pipe when there is a hydraulic jump.

### 4.3.4 Downstream Control (Tailwater) Elevation

The hydraulic analysis of a storm drain system typically begins at the downstream outfall. The controlling water surface elevation at the point of discharge is commonly referred to as the tailwater. At the outfall, one of several conditions will be encountered: another closed storm drain; outfall to a drainage channel, storage facility, reservoir, lake, or detention facility, a free outfall, or a tidally influenced outfall. The tailwater elevation criteria described here are for determining HGL and EGL elevations only.

For free outfalls, the initial water surface elevation (tailwater) shall be assumed equivalent to the soffit elevation. For outfalls into other drainage facilities, a drainage channel, reservoir, or detention facility, the initial water surface elevation shall be set at the 100-year water surface elevation calculated for the downstream facility. If no downstream information is available, the water surface elevations identified on the appropriate Flood Insurance Rate Map (FIRM) at the location of the outfall may be used. In cases where the storm drain outfall condition is tidally influenced, it is usually sufficient to use the mean higher high-water elevation as the tailwater elevation. In cases where storm surge is a concern or for other situations with unusual tailwater conditions, the appropriate design outfall tailwater elevation shall be chosen in consultation with the governing OCPW Department.

## 4.3.5 Energy Loss Calculations

### 4.3.5.1 Friction Loss

Manning's Formula shall be used for determination of friction slope (S) in the Bernoulli equation. Tables, charts, head loss formulas, and calculation sheets included here are based on the Manning's Equation:

$$Q = \frac{1.486}{n}\, A\, R^{2/3}\, S_f^{1/2}$$

Where:

| | | |
|---|---|---|
| $Q$ | = | quantity of water (cfs) |
| $A$ | = | flow area of conveyance section storm drain ($ft^2$) |
| $n$ | = | Manning's coefficient of roughness, See Table 4-1 (Dimensionless) |
| $R$ | = | hydraulic radius (ft) $R = \frac{A}{P}$ |
| $P$ | = | wetted perimeter (ft) |
| $S_f$ | = | friction slope (ft/ft) |

Friction losses for hydraulic sections carrying storm water, including pump station discharge lines, shall be calculated from the Manning Equation.

When the Manning Equation is rearranged into a more useful form,

$$S_f = \left(\frac{Q_n}{1.486\, AR^{2/3}}\right)^2 = \left(\frac{Q}{K}\right)^2$$

In which:

$$K = \frac{1.486\, A\, R^{2/3}}{n}$$

The loss of head due to friction throughout the length of reach (L) is calculated by:

$$h_f = S_f L = \left(\frac{Q}{K}\right)^2 L$$

The value of $K$ is dependent upon only two factors: the geometrical shape of the flow cross section as expressed by the quantity, $AR^{2/3}$, and the roughness coefficient, n.

#### 4.3.5.2 Transition Loss

Most references equate losses in transitions directly to change in velocity head through use of coefficients. This method has recently become debatable and research tacitly indicates a future general adoption of specific force (pressure plus momentum) principles for determining transition losses. However, until the P + M method is further documented, energy coefficients probably represent the most practical criteria for analysis (energy coefficient α and momentum coefficient β). Therefore, losses in transitions shall be determined by:

For velocity increases in transition:

$$h = \frac{K_i(V_2^2 - V_1^2)}{2g}$$

For velocity decreases in transition:

$$h = \frac{K_o(V_2^2 - V_1^2)}{2g}$$

| Shape | $K_i$ | $K_o$ |
|---|---|---|
| Abrupt (Square) | 0.30 | 0.80 |
| Straight Line* 10° | 0.10 | 0.20 |
| Straight Line* 15° | 0.10 | 0.30 |
| Straight Line* 20° | 0.10 | 0.40 |
| Straight Line* 30° | 0.10 | 0.70 |
| Warped Design** | 0.10 | 0.20 |

Notes:
* Angle given is maximum water boundary angle relative to transition centerline.
** Reversing curves with maximum convergent angle of 12°30’ $K_i$ and maximum divergent angle of 5° 45’ for $K_o$.

**Table 4-2: K Values for Calculating Transitional Losses**

#### 4.3.5.3 Junction Loss

There are two methods to compute junction loss. Junction losses shall be calculated by equating pressure plus momentum through the confluences under consideration. This can be done by using either OCPW P + M method or the City of Los Angeles Thompson ΔY equation. Both methods are applicable in all cases for pressure flow and give the equivalent results.

#### 4.3.5.4 Pressure plus Momentum Method

For the special case of pressure flow, friction neglected, the junction head loss can be calculated by:

$$h_j = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} - \left(\frac{2A_3}{A} \times \frac{V_3^2}{2g}\right)\cos\theta$$

Where:

| | | |
|---|---|---|
| $A$ | = | cross section area of flow conveyance in the storm drain (ft$^2$) |
| $g$ | = | gravity, 32.2 (ft/s$^2$) |
| $V$ | = | Velocity (ft/s) |

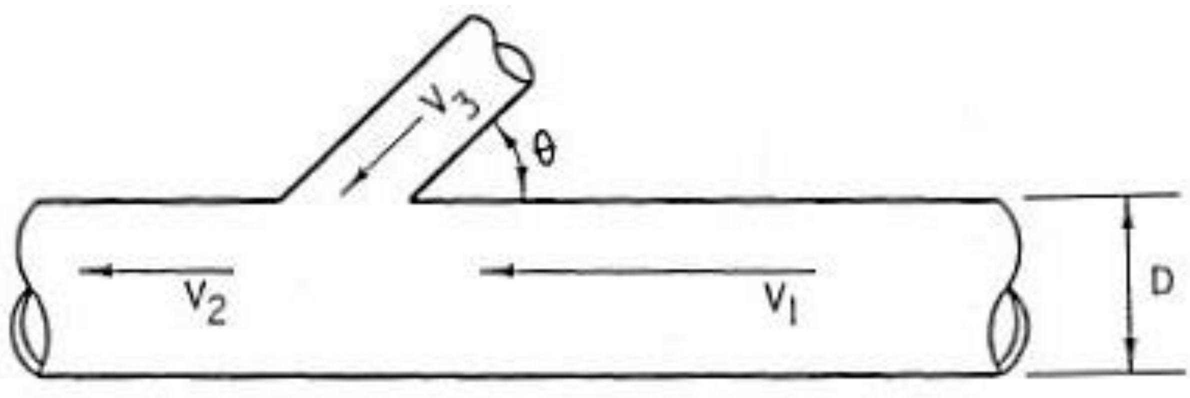



**Figure 4-4: Pressure plus Momentum**

#### 4.3.5.4.1 Thompson Equation Method

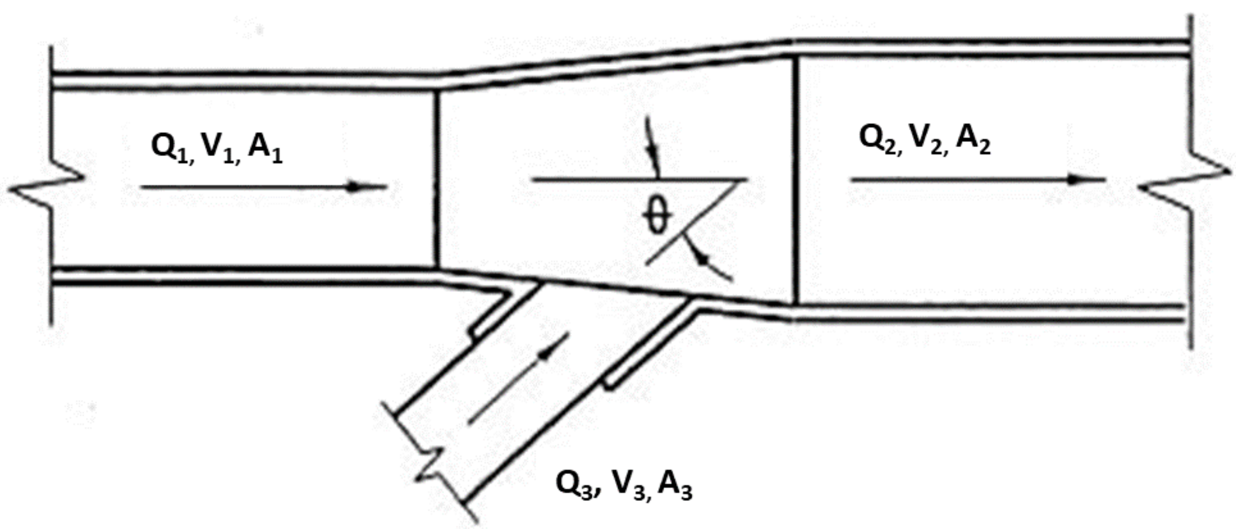



**Figure 4-5: Thompson Equation**

The Thompson Equation for junctions is described by the following:

$$\Delta y \times A_{avg} = \frac{Q_2V_2 - Q_1V_1 - Q_3V_3\cos\theta_3}{g}$$

Where:

| | | |
|---|---|---|
| $\Delta y$ | = | Difference in hydraulic gradient for the two end sections (ft) |
| $A_{avg}$ | = | Average Area $= 1/6\ (A_1 + 4A_m + A_2)$ or, for practical use $1/2\ (A_1 + A_2)$ (ft$^2$) |
| $A_m$ | = | Mean area of flow (ft$^2$) |
| $Q$ | = | quantity of water (cfs) |
| $g$ | = | gravity, 32.2 (ft/s$^2$) |
| $V$ | = | Velocity (ft/s) |
| $\theta$ | = | angle of pipe (°) |

The above equation is applicable only to prismoidal and circular conduits or channels. The friction force may be considered negligible or can be calculated and taken into account. The Thompson equation shall not be used when an open channel changes side-slope going through a junction. A gain in energy will occur when areas change drastically, use of Thompsons $\Delta y$ in these cases should be avoided.

The angle of confluence between main line and lateral shall not exceed 30 degrees except under the following conditions:

1. The flow (Q) in the proposed lateral does not exceed 10 percent of the main line flow, and
2. The size of the lateral is 60" (20 $ft^2$) or less.
3. The hydraulic calculations do not indicate excessive head losses occurring in the mainline storm drain due to the confluences.

#### 4.3.5.5 Manhole Loss

Manhole losses shall be calculated from the equation shown below and shall be used only for no change in Q and no change in pipe sizes. Where a change in pipe size and/or change in Q occurs, the head loss shall be calculated in accordance with Transition Loss and Junction Loss.

$$h_m = 0.05 \frac{V^2}{2g}$$

#### 4.3.5.6 Bend Loss

Bend losses shall be calculated from the following equations:

$$h_b = K_b \frac{V^2}{2g}$$

In which:

$$K_b = 0.25 \sqrt{\frac{\theta}{90}}$$

Where:

| | | |
|---|---|---|
| g | = | gravity, 32.2 ($ft/s^2$) |
| V | = | Velocity (ft/s) |
| θ | = | Central angle of bend, not to exceed 90 degrees (°) |

Bend losses should be included for all closed pipe, those flowing partially full as well as those flowing full.

#### 4.3.5.7 Angle Point Loss

Angle point losses shall be calculated from the following equation:

$$h_{apt} = 0.02\frac{V^2}{2g}$$

## 4.4 Pipe Types and Criteria

### 4.4.1 General

#### 4.4.1.1 Life of Structures

The basis for structural design shall be a design life of 100 years for all permanent drainage structures within the County, except where otherwise noted for specific pipe types.

In selection of the structural section, the factors to be considered include hydraulics, debris, maintenance, safety, traffic, R/W, property, economics and aesthetics.

#### 4.4.1.2 Multiple Storm Drain

Multiple pipes (branching of flow) or parallel storm drain systems are not permitted in new facilities.

#### 4.4.1.3 Storm Drain Designations

Table 4-3 indicates the pipe designations that shall be used on improvement plans. This list of pipe designation shall be used as a reference only. It lists several pipes that are not acceptable for use as public storm drain in Orange County. Please consult the sub-sections in Section 4.4 for types of pipe accepted in Orange County. See Section 4.2.14 for information on allowable materials.

| Designation | Conduit |
|---|---|
| ABS | Acrylonitrile – Butadiene- Styrene Composite Sewer Pipe** |
| ACP | Asbestos-Cement Pipe* |
| CAP | Corrugated Aluminum Pipe*** |
| CAPA | Corrugated Aluminum Pipe Arch*** |
| CASP | Corrugated Aluminized Steel Pipe* |
| CIPCP | Cast-in-Place Non-reinforced Concrete Pipe*** |
| CMP | Corrugated Metal Pipe*** |
| CMPA | Corrugated Metal Pipe Arch*** |
| CMPC | Corrugated Metal Pipe Bituminous Coated (dipped)*** |
| CSP | Corrugated Steel Pipe*** |
| CSPP | Corrugated Steel Plate Pipe |
| CMPCL | Corrugated Metal Pipe Bituminous Coated and Lined |
| CMPI | Corrugated Metal Pipe Paved Invert*** |
| CMPL | Corrugated Metal Pipe Fully Bituminous Coated*** |
| HDPE | High Density Polyethylene Pipe* |
| PACP | Perforated Asbestos-Cement Pipe** |
| PAP | Perforated Aluminum Pipe** |
| PCLP | Perforated Clay Pipe** |
| PP | Polypropylene Pipe (AASHTO Designation M330-20)* |
| PPVCP | Perforated Polyvinyl Chloride Pipe**, *** |
| PVCP | Polyvinyl Chloride Pipe*, **,*** |
| RCA | Reinforced Concrete Arch |
| RCB | Reinforced Concrete Box |
| RCP | Reinforced Concrete Pipe |
| RCPA | Reinforced Concrete Pipe Arch |
| SRP | Spiral Rib Pipe |
| SSPA | Structural Steel Plate Arch |
| SSPPA | Structural Steel Plate Pipe Arch |
| VCP | Vitrified Clay Pipe* |

Notes:
* These storm drains are not approved use as public storm drains in Orange County.
** Subsurface seepage drainage only; not approved for use as public storm drains.
*** Not to be used longitudinally in arterial highways.

**Table 4-3: Storm Drain Designations**

### 4.4.1.4 Permissible Velocities

Permissible Velocities shall follow the Caltrans Highway Design Manual Table 855.2A (Caltrans, 2018).

#### 4.4.1.5 Minimum Cover

Minimum cover for various pipe materials is shown in Table 4-4 which has been adapted from the Caltrans Highway Design Manual (Caltrans, 2018, Table 856.5). The minimum pipe cover depth is 30”.

| Minimum Thickness[1] of Cover at Edge of Traveled Way for HL-93 Live Load Conditions | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Corrugated Metal Pipes and Pipe Arches** | **Steel Spiral Rib Pipe** | **Aluminum Spiral Rib Pipe $S^2 \leq 48''$** | **Aluminum Spiral Rib Pipe $S \geq 48''$** | **Structural Plate Pipe** | **Reinforced Concrete Pipe (RCP) under Rigid Pavement** | **RCP Under Flexible Pavement or Unpaved** | **Plastic Pipes** |
| S/8 or 30” Min | S/4 or 30” Min | S/2 or 30” Min | S/2.75 or 30” Min | S/8 or 30” Min | 30” Min | (Max Outside Dimension)/8 or 30” Min | S/2 or 30” Min |

[1] Minimum thickness of cover is measured at ultimate or failure edge of traveled way
[2] “S” is the maximum inside diameter or span of a section.

**Table 4-4: Minimum Thickness of Cover over Pipe**

### 4.4.2 <u>Reinforced Concrete Pipe (RCP)</u>

#### 4.4.2.1 General

RCP is built from Portland cement concrete and reinforcing steel in a variety of shapes, sizes, and lengths. Two types of RCP predominate: “Spun RCP” and “Dry cast RCP”.

#### 4.4.2.2 Areas of Use/Limitations

#### 4.4.2.3 Curves

The minimum radius for pipe centerline curves shall be per OCPW Standard Plan 380-4-OC and manufacturers recommendations. Deflection angle shall not exceed maximum as shown per OCPW Standard Plan 380-4-OC without prior approval from OCPW. Large angle points will increase constant “0.02”.

#### 4.4.2.4 Limitations on Use of Concrete by Acidity of Soil and Water

Table 4-5 and Table 4-6 show the required concrete mixes for sulfate resisting concrete.

| 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 | Caltrans Standard Specifications Section 90 | No Restrictions |
| 5.6 to 7.0 | 1,500 to1,999 | Caltrans 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 4-5: Guide for the Protection of Cast-In-Place and Precast Reinforced and Unreinforced Concrete Structures (5) Against Acid and Sulfate Exposure Conditions (1), (2) (Caltrans, 2018, Table 855.4A)**

| Water-Soluble Sulfate in Soil Sample1 (Percent) | Sulfate in Water Sample1 (Parts Per Million) | Type of Cement | Cement Factor2 |
|---|---|---|---|
| 0-0.20 | 0-2,000 | II | Minimum required by Specifications |
| 0.20-0.50 | 2,000-5,000 | V | Minimum required by Specifications |
| | | II | 7 Sacks/cubic yard |
| 0.50-1.50 | 5,000-15,000 | V | Minimum required by Specifications |
| | | II | 7 Sacks/cubic yard |
| Over 1.50 | Over 15,000 | V | 7 Sacks/cubic yard |

Notes:
1) Reported as $SO_4$. If both conditions apply use highest level in table.
2) Recommended measures for type and amount of cement based on analysis of sulfate content in soil and water.

**Table 4-6: Guide for Sulfate Resisting Concrete Pipe**

### 4.4.2.5 Slope and Velocity Limitations

1. RCP installed on slopes over twenty percent (20%) shall have water-tight joints, reinforced masonry or reinforced cast in place PCC cutoff walls to reduce leaks and potential piping.
2. Slope anchors will be installed per Section 4.2.10.
3. Velocity shall not exceed 20 fps in standard wall RCP.
4. Where velocity exceeds 20 fps, a special wall RCP with a minimum of 1 1/2-inch steel clearance on the inside surface shall be used.
5. Maximum velocity in special cover RCP shall be 40 fps for clear flow and shall be less than 15 fps for heavy bed load. Velocities higher than 40 fps shall require OCPW Engineer's approval.
6. Velocity rings shall be provided where grade of pipe exceeds 40% and lengths of pipe exceeds 100' unless velocities show that they are not required. Velocity rings shall not be metal band type but shall be cast-in-pipe type and provide a low flow pass through notch. See Section 8.3.3, "Dissipator Rings," for more information.

### 4.4.2.6 High Ground Water

In areas where a high-water table exists, a special study shall be performed and a recommendation on sealed joints, water quality, and groundwater depletion shall be included.

### 4.4.2.7 Cover Over Pipe

The cover is measured as the distance from the finished grade to the outside top of the pipe or culvert. When minimum covers are encountered on drainage design projects, the design requirements for the pipes shall be as follows:

1. Use of an at-grade reinforced concrete box (see Chapter 5 for RCB) is recommended in place of RCP when top of culvert is within 1' of grade (see Orange County Flood Control Design Manual).
2. When cover is 1' or greater determine pipe strength using the D-Load Calculations included in Chapter 10.2.7 (Live loads with impact shall be included as specified).
3. D-Load Calculations are required and included in Section 10.2.7.

### 4.4.2.8 Junction Structures

Reinforced Concrete Pipe shall use the standard junction structures found in the OCPW Standard Plans.

### 4.4.2.9 Special Provision for Steel Cover

1. Cover is defined as the amount of concrete cover over the reinforcing steel on the inside face of the pipe.
2. Steel cover or "Extra Cover" is a feature that is unique to reinforced concrete pipe. It is the amount of additional concrete cover requested in place of the minimum cover of 1/2" required by the standard specification.
3. The wall thickness of a pipe may be increased to accommodate soil or structural challenges, such as exceptionally high D-loads that require more steel, or sulfate corrosion needs, or abrasion potential.

### 4.4.3 Reinforced Concrete Box (RCB)

Reinforced Concrete Box criteria can be found in Chapter 5 Culverts

### 4.4.4 Corrugated Steel Pipe (CSP, Type 5 Inlet Structures)

#### 4.4.4.1 General

A maximum life of 50-years shall be used for CSP per Figure 4-6: Minimum Thickness of Metal Pipe for 50-year Maintenance-Free Service Life (Caltrans, 2018, Figure 855.3A), unless added to by Table 4-7. Additional covering or dipping for velocity or soils conditions shall not be used in alternate calculations exceeding 50-year life.

#### 4.4.4.2 Areas of Use/Limitations

The following paragraphs discuss the limitations and areas of use for CSP.

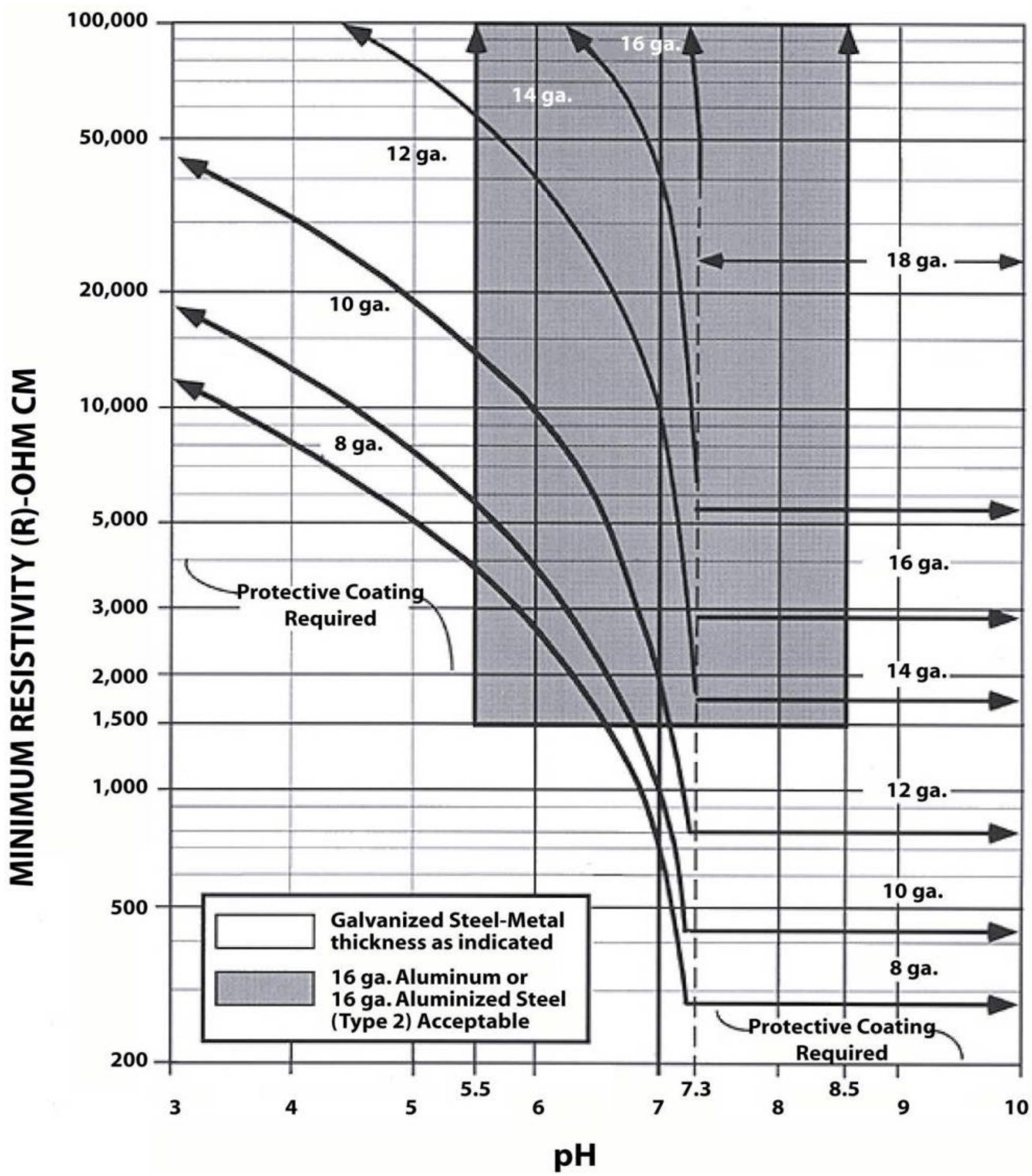
#### 4.4.4.3 Placement

Table 4-8 and Table 4-9 show the gauge requirements for CSP. CSP shall not be placed longitudinally under the structural section of arterial or major highways except in temporary installation. The depth of cover should not exceed 20’ except in street crossings of natural streams. CSP may be used for crossing of natural streams and in habitat areas where large culverts are required. Depths of cover exceeding 20’ will require special OCPW approvals and provide for repair/replacement by either jacking of new reduced size culvert or concrete lining.

Structural strength of CSP is based on lateral support; therefore, it should not be used where adjacent facilities are expected to be removed/replaced within a reasonable life of the pipe.

#### 4.4.4.4 Acceptance for Maintenance

CSP will not be accepted for maintenance by Orange County or OCFCD for any portions, extensions or connections to permanent storm drains. Temporary facilities or extensions that will exist for less than 10 years are accepted but will require prior approval by OCPW.  The minimum thickness of metal pipe for fifty-year maintenance free service life is shown in Figure 4-6.




Notes:
For pH and resistivity levels not shown refer to California Test 643.

**Figure 4-6: Minimum Thickness of Metal Pipe for 50-year Maintenance-Free Service Life (Caltrans, 2018, Figure 855.3A)**

| Flow Velocity (fps) | 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 bed loads | **** | **** | **** | **** |

* 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 OCPW.
Notes:
(1) Where there are increased velocities with minor bed load volumes, much higher velocities may be applicable.
(2) Range of additional service life commensurate with flow velocity range.
(3) See Section 4.4.4.5 for Pipe Durability

**Table 4-7: Guide for Anticipated Service Life Added to Steel Pipe by Protective Coating (Caltrans, 2018, Table 855.2C)**

| Diameter (inches) | Maximum Height of Cover (feet) | | | | | |
|---|---|---|---|---|---|---|
| | Metal Thickness | | | | | |
| | 18 gage 0.052” | 16 gage 0.064” | 14 gage 0.079” | 12 gage 0.109” | 10 gage 0.138” | 8 gage 0.168” |
| 12 | 118 | 148 | 177 | | | |
| 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 |

**Table 4-8: Maximum Cover Height for Corrugated Steel Pipe (2-2/3”x1/2” Helical Corrugations) (Caltrans, 2018, Table 856.3A)**

| Diameter (Inches) | Maximum Height of Cover (feet) | | | | |
|---|---|---|---|---|---|
| | Metal Thickness | | | | |
| | 16 gage 0.064” | 14 gage 0.079” | 12 gage 0.109” | 10 gage 0.138” | 8 gage 0.168” |
| 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 4-9: Maximum Cover Height for Corrugated Steel Pipe (3”x1” Helical Corrugations) (Caltrans, 2018, Table 856.3A)**

#### 4.4.4.5 Pipe Durability

Table 4-7 constitutes a guide for estimating the added service life that can be achieved by bituminous coatings and invert paving for steel pipes. The guide values for years of added service life may 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 4-7:

1. Channel Materials: If there is no existing culvert, it should be assumed that channel material 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. For continuous flow, the years of invert protection can be expected to be decreased from Table 4-7.

2. Asphalt mastic or polymeric coatings are alternatives for non-abrasive flow conditions on the inside of the culvert. Under these circumstances, a special provision will be required to specify this alternative.

#### 4.4.4.6 Structural Criteria

The use of CSP shall be based upon the following structural requirements:

#### 4.4.4.7 Corrosive Environments

CSP is subject to attack from corrosive environments. Service life can be extended by using heavier gauges, bituminous coating and paving the invert of the pipe or cathodic protection. Soil analysis and corrosion analysis to establish necessary gauge thickness and treatments to obtain the 50-year service life will be required with first plan check submittal.

#### 4.4.4.8 Soil Tests for Metal Drainage Storm Drain

Use of any metal storm drain will require reports on the following:

a. Soil resistivity and pH values along the proposed drain location

b. pH and resistivity of ground water and low flows

c. Backfill soil resistivity and pH

d. History and present condition of existing storm drain in the watershed area if any exist

Resistivity and pH tests shall be conducted by a County-approved testing laboratory to determine the minimum resistivity and pH values.

Figure 4-7 shows relationships between resistivity and years to perforation for an 18-gage steel culvert and conversion factors for other gages.

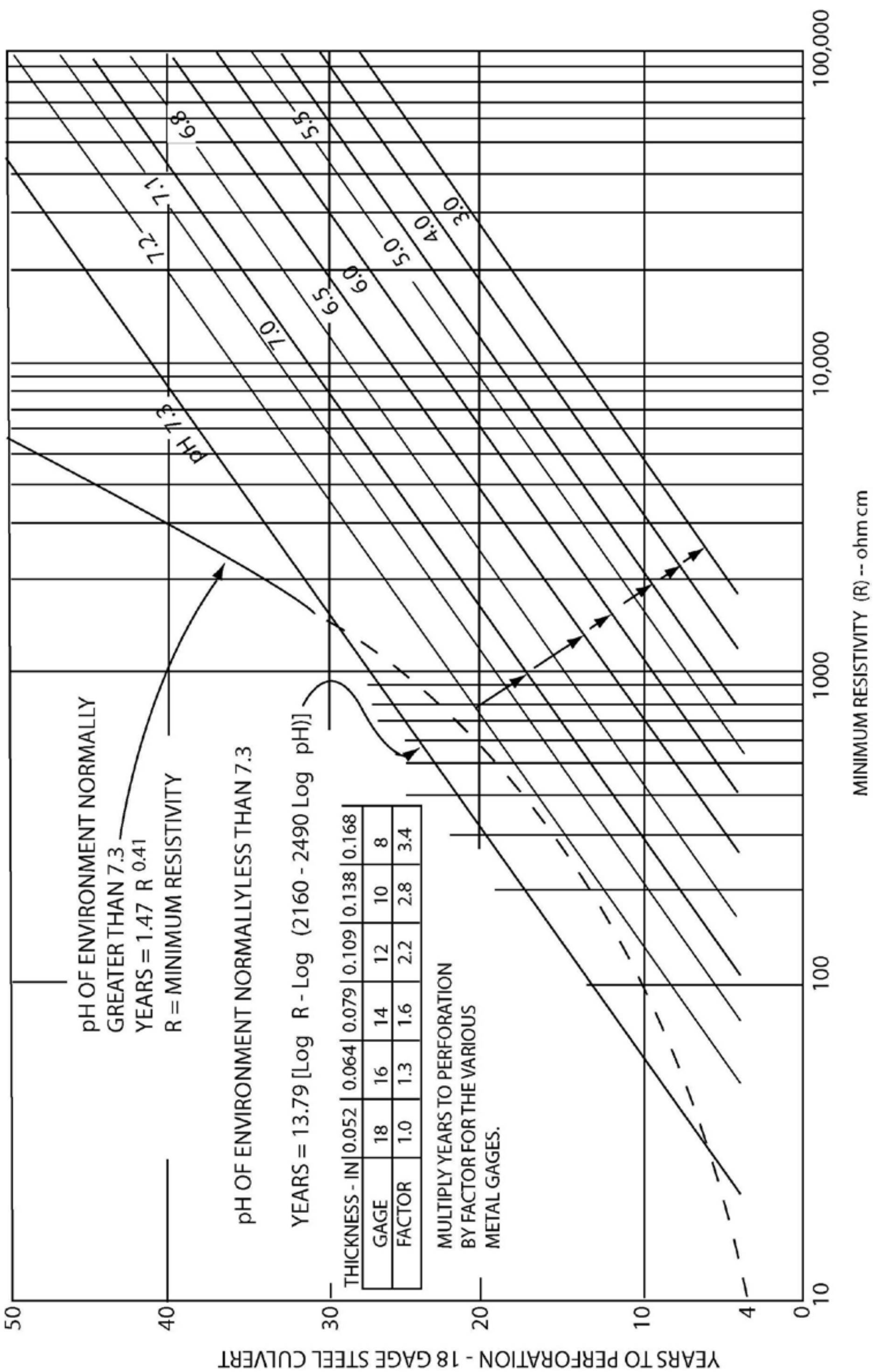



**Figure 4-7: Estimated Years to Perforation of Steel Culverts (Caltrans, 2018, Figure 855.3B)**

### 4.4.4.9 High Velocity Flows

Neither galvanizing nor asphalt coating sufficiently protects the base metal under high velocity flow. Thicker gauges or proprietary coatings must be used, and a cost comparison with RCP shall be performed for annuity calculation purposes.

### 4.4.4.10 Service Life

The Department of Transportation (Caltrans) method shall be used to determine the service life of culverts. See California Test 643 (1978) and chapter 850 section 855 in the Caltrans Highway Design Manual (Caltrans, 2018).

### 4.4.4.11 Minimum Cover for CSP

Recommended minimum height of cover in feet for corrugated steel pipe is shown in Table 4-10. During installation of the structure, backfilling operations create side pressures that tend to vertically elongate the pipe. As filling progresses above the top of the pipe, a point is reached where the vertical load exactly equals the active lateral pressure on the pipe and the structure then becomes a compression ring. The practical fill level over the pipe is something less than the theoretical height and is summarized in here. The engineer shall refer to the manufacturer's installation instructions.

| Diameter (inches) | 16 gage Cover (ft) | 14 gage Cover (ft) | 12 gage Cover (ft) | 10 gage Cover (ft) | 8 gage Cover (ft) |
|---|---|---|---|---|---|
| 15 | 1.0 | 1.0 | | | |
| 18 | 1.0 | 1.0 | 1.0 | | |
| 21 | 1.0 | 1.0 | 1.0 | | |
| 24 | 1.0 | 1.0 | 1.0 | 1.0 | |
| 30 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| 36 | 1.5 | 1.0 | 1.0 | 1.0 | 1.0 |
| 42 | 1.5 | 1.0 | 1.0 | 1.0 | 1.0 |
| 48 | 1.5 | 1.0 | 1.0 | 1.0 | 1.0 |
| 54 | (1.5) | 1.0 | 1.0 | 1.0 | 1.0 |
| 60 | (2.0) | (1.5) | 1.0 | 1.0 | 1.0 |
| 66 | (2.0) | (1.5) | 1.0 | 1.0 | 1.0 |
| 72 | (2.0) | (1.5) | (1.0) | 1.0 | 1.0 |
| 84 | | (1.5) | (1.0) | (1.0) | 1.0 |

Notes:

- Minimum pipe stiffness requirements for practical handling and installation are based on resultant flexibility factor FF and limits the size of each combination of corrugation and metal thickness.
- Pipe sizes for figures shown in parenthesis indicate diameter gage combinations that lie beyond the recommended flexibility factor.

Flexibility factor given by equation: $FF = \frac{D^2}{EI}$

Where: $E$ = modulus of elasticity = 30 x $10^6$ psi
$D$ = diameter or span in"
$I$ = moment of inertia of wall in $in^4$

**Table 4-10: Recommended Minimum Cover for CSP**

### 4.4.4.12 Elbow/Junction Structures/Confluence

Configurations for elbows and other junctions for CSP are shown in Figure 4-8. Recommended CSP manhole dimensions per California Corrugated Steel Pipe Association are shown in Figure 4-9. A table of standard CSP fitting dimensions is provided (Table 4-11).

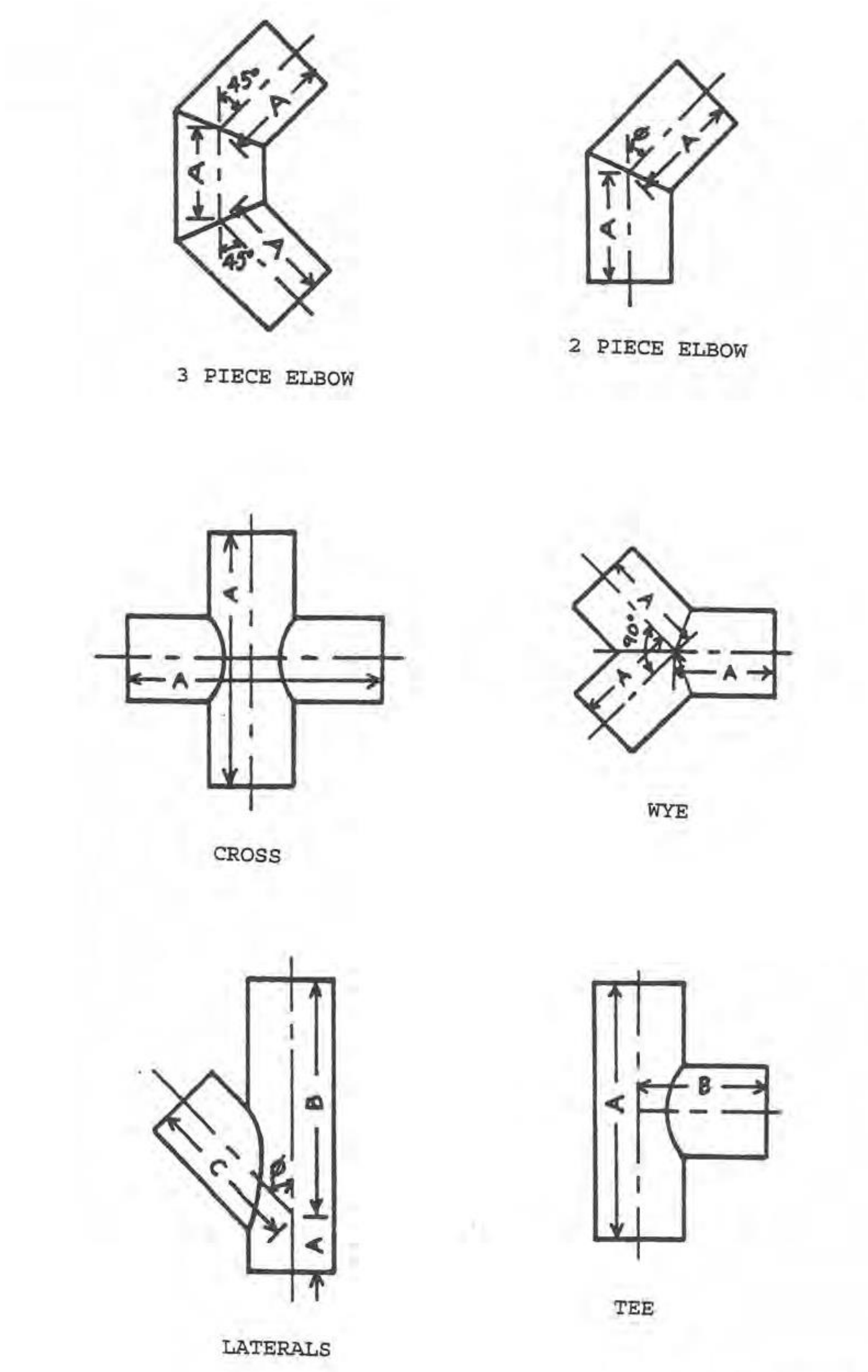



**Figure 4-8: Corrugated Steel Pipe Elbow/Junction/Confluence Details**

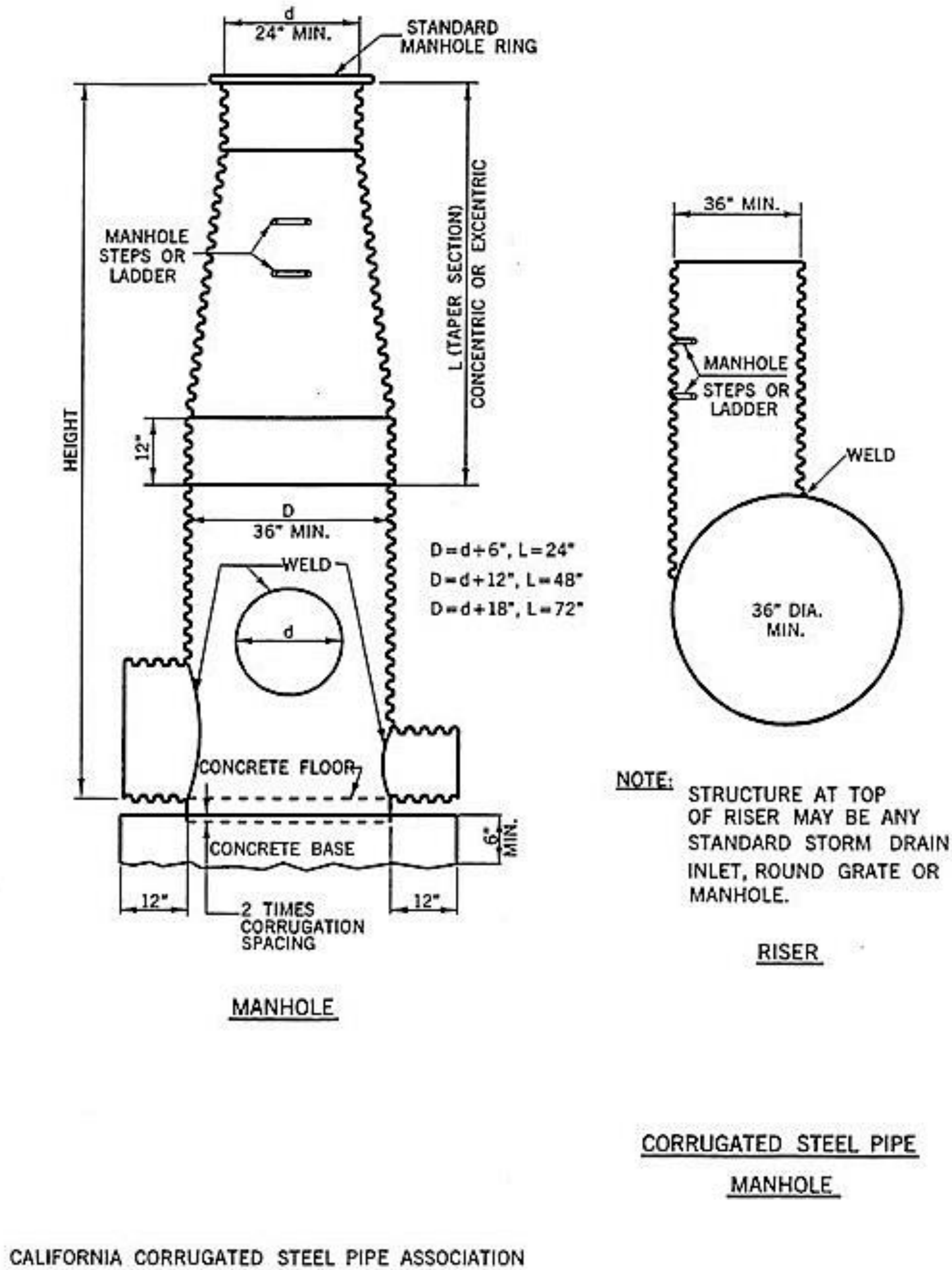



**Figure 4-9: Corrugated Steel Pipe Manhole**

| Pipe Diameter (inches) | 2 Piece Elbow ϕ= 0°- 90° | 3 Piece Elbow | Cross | Wye | Tee[1] | | Lateral Main & Branch same dia. ϕ= 45° [1,2] | | |
|---|---|---|---|---|---|---|---|---|---|
| | A | A | A | A | A | B | A | B | C |
| 6 | 12 | 16 | 24 | 24 | 24 | 12 | 8 | 16 | 24 |
| 8 | 12 | 16 | 24 | 24 | 24 | 12 | 8 | 16 | 24 |
| 10 | 12 | 16 | 24 | 24 | 24 | 12 | 12 | 24 | 36 |
| 12 | 24 | 16 | 24 | 24 | 24 | 12 | 12 | 36 | 36 |
| 15 | 24 | 16 | 36 | 24 | 36 | 24 | 12 | 36 | 36 |
| 18 | 24 | 16 | 36 | 24 | 36 | 24 | 12 | 36 | 48 |
| 21 | 24 | 16 | 36 | 24 | 36 | 24 | 12 | 36 | 48 |
| 24 | 24 | 16 | 48 | 24 | 48 | 24 | 24 | 36 | 48 |
| 30 | 24 | 16 | 48 | 24 | 48 | 36 | 24 | 48 | 60 |
| 36 | 36 | 24 | 48 | 36 | 60 | 36 | 36 | 48 | 72 |
| 42 | 36 | 24 | 72 | 36 | 60 | 36 | 36 | 60 | 72 |
| 48 | 36 | 24 | 72 | 36 | 72 | 36 | 36 | 84 | 84 |
| 54 | 48 | 32 | 96 | 48 | 72 | 48 | 36 | 96 | 96 |
| 60 | 48 | 32 | 96 | 48 | 96 | 48 | 36 | 96 | 108 |
| 66 | 48 | 32 | 96 | 48 | 96 | 48 | 36 | 108 | 108 |
| 72 | 60 | 40 | 120 | 60 | 120 | 60 | 36 | 108 | 120 |
| 78 | 60 | 40 | 120 | 60 | 120 | 60 | | | |
| 84 | 60 | 40 | 120 | 60 | 120 | 60 | | | |
| 90 | 60 | 40 | 120 | 60 | 120 | 60 | | | |
| 96 | 72 | 48 | 132 | 72 | 132 | 72 | | | |

1 Branch may be fabricated from any dia. pipe up to dia. of line pipe.
2 Branches may be fabricated with any ϕ angle from 30° to 90°.
Notes: All variations from Table are special designs
See Figure 4-8 for Junction /Confluence drawings
See Figure 4-9 for CSP Manhole Schematic.

**Table 4-11: Minimum Dimensions for Standard CSP Fittings**

## 4.4.5 Corrugated Steel Plate Pipe (CSPP)

### 4.4.5.1 General

Corrugated steel plate is also referred to as structural plate pipe and is field constructed by bolting plates together to create a structural arch. The 6” x 2” corrugation is the current standard used by AASHTO and Caltrans and is hereby adopted by this manual.

### 4.4.5.2 Areas of Use/Limitations

Corrugated steel plate arch or structural plate may be used in rural settings, for trail or wildlife crossing, to achieve a comfortable width for passage of wildlife and shall not be used in urban or commercial areas where upstream or downstream concrete channels are existing or proposed.

### 4.4.5.3 Curves

Curves exceeding 2 degrees of curvature shall not be allowed without a special design and fabrication of special plates. Field cutting of plates is prohibited.

#### 4.4.5.4 Maintenance Issues

The performance of a flexible culvert is dependent on soil structure interaction, stiffness, and chemical composition of the soil. The design shall consider and provide for all these features. Side/backfill shall be a material with little or no plasticity and free from organic material. Caltrans specifications for steel plate structures shall be used (Caltrans, Standard Specifications Latest Edition).

#### 4.4.5.5 Junction Structures/Manholes

Junction manhole structures shall be shop fabricated and shall provide for access per OCPW Standard Plans for RCP installation of compatible sizes.

#### 4.4.5.6 Height-of-Fill

The maximum height of cover over structural plate pipe and pipe arches with 6” by 2” corrugations for available diameters and thicknesses are shown in Table 4-12 and Table 4-13, respectively. The following criteria apply to these tables:

1) The values given for each size of structural plate pipe or arch constitute the maximum height of cover for the thickness of metal.
2) The thickness shown is the structural minimum. Where abrasive conditions are anticipated, additional metal thickness or a paved invert shall be provided.
3) Adequate provisions for corrosion resistance shall be made to achieve the required service life.
4) The limits of heights of cover for structural plate arches are based on the supporting soil sustaining bearing pressures of 1and 3 tons per square foot at the corners.

| Maximum Height of Cover (ft) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Diameter (inches)** | **Metal Thickness (inches)** | | | | | | | |
| | **0.110 (12 gage)** | **0.140 (10 gage)** | **0.170 (8 gage)** | **0.218 (5 gage)** | **0.249 (3 gage)** | **0.280 (1 gage)** | **0.318 (0 gage)** | **0.380 (000 gage)** |
| 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 |
| 78 | 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 4-12: Structural Steel Plate Pipe (6” x 2” Corrugations) Maximum Cover (Caltrans, 2018, Table 856.3M)**

| Maximum Height of Cover (ft) | | | |
|---|---|---|---|
| **Span** | **Rise** | **Factored Corner Soil Bearing – 6 tons/ft$^2$** | |
| | | **Metal Thickness (in)** | |
| | | **0.110 (12 gage)** | **0.138 (10 gage)** |
| | | **18" Corner Radius** | |
| 6'-1" | 4'-7" | 21 | |
| 7'-10" | 5'-1" | 18 | |
| 7'-11" | 5'-7" | 16 | |
| 8'-0" | 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" corner radius arch should be specified when conditions will permit its use.
(3) The limit of heights of cover for structural plate arches based on the supporting soil sustaining bearing pressures of 1and 3 tons per square foot at the corners.

**Table 4-13: Structural Steel Plate Pipe Arches (6" x2" Corrugations) Maximum Cover (Caltrans, 2018, Table 856.3N)**

### 4.4.6 Plastic Pipe (Sub drainage and Irrigation)

Plastic pipe may be used for sub drainage and irrigation and shall not exceed 36" in diameter. This section covers the following types of plastic pipe:

1. Acrylonitrile – butadiene – styrene (ABS)
2. Polyvinyl chloride (PVC) Pipe, Plastic Pipe
3. ABS, Solid Wall Pipe
4. Polyethylene (PE) Solid Wall Pipe
5. Corrugated PE Pipe with Smooth Interior

#### 4.4.6.1 General Criteria

An alternative life of 50 years shall be used.

#### 4.4.6.2 Areas of Use/Limitations

Plastic pipe may be used for drainage applications provided the following conditions are met:

1. Maximum cover shall be 20' unless otherwise authorized by the design engineer.
2. Minimum diameter shall be 4".
3. Maximum diameter shall be 36".
4. Plastic pipe shall not be used to drain an arterial highway. Plastic pipe shall not be used within the R/W of an arterial highway, except for the following applications:
    a. Landscape median drains
    b. Sub drains
5. Minimum cover within streets shall be 30".
6. Plastic pipe in streets shall use slurry backfill in accordance with Caltrans Specifications.
7. Plastic pipe outside streets shall use slurry backfill in accordance with Caltrans Specifications when pipe is greater than 18" in diameter.
8. Plastic pipe outside streets shall use slurry backfill in accordance with Caltrans Specifications when cover is less than 30" and pipe is subjected to highway loading.

### 4.4.7 Polypropylene (PP) and High Density Polyethylene (HDPE) Pipe

#### 4.4.7.1 General

The standards regarding the durability of PP (AASHTO Designation M 330-20) and HDPE are based on the long-term performance of its material properties. See Caltrans Highway Design Manual Index 852.6(2)(a) Strength Requirements for allowed materials and wall profile types. PP and HDPE exhibit good abrasion resistance and are virtually corrosion free.

The primary environmental factor limiting service life of PP and HDPE is ultraviolet (UV) radiation from sunlight exposure. Current testing on HDPE products conforming to specification requirements for including carbon black as a component of HDPE pipe, have exhibited adequate UV resistance. For the sake of safeguarding against ignition, both types of pipes shall minimize termini exposure to UV in locations and situations for which they are permitted as installations.

Some Polypropylene Pipe (PP) can be used as an alternative to HDPE. Based on the manufacturer, isotactic PP may exhibit higher purity, strength, rigidity, and abrasion resistance than HDPE (Hoppe 2011). Since PP is a relatively new product manufactured by a single manufacturer or a couple of manufacturers, this manual will not provide any specific guidance for its use. However, if a design engineer wishes to utilize a PP product the design engineer will be required to get approval from OCPW.

### 4.4.7.2 Areas of Use/Limitations

Where high fire danger exists, the designer shall use RCP in lieu of PP and HDPE. Plastic pipes and pipes with coatings (typically of bituminous or plastic materials) are the most susceptible to damage from fire. HDPE can continue to burn as long as an adequate oxygen supply is present. The rate of burning is fairly slow and often self-extinguished if the airflow is inhibited (i.e., pipe not aligned with prevailing wind or ends sheltered from air flow). The canyons in Orange County typically have brush fires on one side of the road resulting in a chimney effect within the pipe. PP and HDPE shall not be used there (see Figure 4-10).

**Figure 4-10: A plastic pipe after a fire near Santiago Canyon Rd in 2020.**

Up to 60-inch diameter of Polypropylene (PP) and High-Density Polyethylene (HDPE) pipe may be installed within private streets and private basins. HDPE and PP shall be corrugated exterior, Type “S” (smooth interior) pipe for storm drain installation as shown on the plans. HDPE and PP pipes that are greater than 30 inches in diameter and accessible by the public shall not be used within 100 feet of the access point, unless otherwise approved by OCPW. Use is restricted in areas with running groundwater or unstable trench walls. In coastal areas or areas with high groundwater, buoyancy shall be checked and verified by the designer.

### 4.4.7.3 Height-of-Fill

Pipe cover over 30 inches shall conform to the California Standard Specifications for Public Works Construction (“Green book”) standards (Public Works Standards, Inc, 2009) for backfill. Pipe cover shall not be less than 30 inches. The allowable fill over PP and HDPE is shown in Table 4-14.

| Size (in) | Maximum Height of Cover (ft) | |
|---|---|---|
| | PP | HDPE |
| 12 | 20 | 15 |
| 15 | 20 | 15 |
| 18 | 20 | 15 |
| 24 | 20 | 15 |
| 30 | 20 | 15 |
| 36 | 15 | 15 |
| 42 | 15 | 15 |
| 48 | 15 | 15 |
| 54 | 15 | 15 |
| 60 | 15 | 15 |

**Table 4-14: Maximum Cover over PP and HDPE Corrugated Pipe Type S (Public Works Standards, Inc, 2009)**

### 4.4.8 Sub Drains

#### 4.4.8.1 General

When there is evidence of excessive groundwater as determined by a geotechnical engineer, a geotechnical report shall be obtained. Solutions may include cutoff trenches, French drains, perforated pipe with gravel material, or vertical wells drilled into the pervious strata.

#### 4.4.8.2 Areas of Use/Limitations

See Orange County Grading Code.

### 4.4.9 RCP Alternate Pipe

The use of products other than RCP must provide for a 100-year life expectancy.

## 4.5 Inlet and Outlet Design

### 4.5.1 Inlet Structures

An inlet structure shall be provided for storm drains located in natural channels. The structure should generally consist of a headwall, wing walls to protect the adjacent banks from erosion, and a paved inlet apron. The apron slope should be limited to a maximum of 2:1. Wall heights should conform to the height of the water upstream of the inlet and be adequate to protect both the fill over the drain and the embankments. Protective barriers shall be provided to prevent public entry. Trash racks should be used for inlets 36-inches (diameter or width) and smaller. For inlets larger than 48 inches, a specially designed trash rack may be required. If debris is prevalent (see Chapter 9), barriers consisting of vertical 3-inch or 4-inch diameter steel pipe spaced at 1/3 the main line diameter or width to a maximum of 30" on centers should be embedded in concrete immediately upstream of the inlet apron. OCPW shall be consulted before design of any high debris storm drain.

### 4.5.2 Outlet Structures

Where conduits or channels discharge into an improved earthen or natural channel, measures must be taken to prevent erosion, head cutting, and property damage. For outlet velocities up to 20 fps, outlet scour protection alone may be considered, but where outlet scour protection cannot be shown to sufficiently reduce velocities to prevent erosion, a suitable energy dissipator shall be installed to reduce discharge to non-erosive velocities. See Chapter 6, "Open Channels", for recommended maximum permissible velocities for unlined channels.

Where a storm drain discharges into the ocean or on to a beach, the Design Engineer shall review preliminary design with OCPW to ensure current criteria as to location and type of structure to be used (the local coastal plan shall be referenced). The structure shall consider the "historical high" beach level as well as the historic eroded beach level.

### 4.5.3 Protective Barriers

Protective barriers shall be provided wherever it is necessary to prevent unauthorized access to storm drains. Typical public facilities where these structures are used include park, open space, or areas where the public may be present at large inlets and outlet storm drain.

Protective barriers shall prevent people from entering storm drains. Protective barriers may consist of, but are not limited to, large, heavy breakaway gates, single horizontal bars across catch basin openings, or chain-link fencing around an inlet or an exposed outlet (see Section 9.4 Hydraulic Design of Debris Basins and Barriers).

## 4.6 Acceptable Hydraulics Software

1. Water Surface Pressure Gradient for Windows (WSPGW)
2. Federal Highway Administration (FHWA) Hydraulic Toolbox
3. Advanced Engineering Software (AES)
4. AutoCAD

If a computer application other than listed above is used, documentation needs to be provided to OCPW with justification for its use.

## 4.7 References

California Department of Transportation. (2018). *Highway Design Manual: U.S. Customary Units (6th Edition)*. https://dot.ca.gov/programs/design/manual-highway-design-manual-hdm/chapter-400-manual-highway-design-manual-hdm

California Department of Transportation. (2018). *Standard Specifications*.

County of Orange Public Facilities and Resources Department. (2000). *Orange County Flood Control District Design Manual*. https://ocip.ocpublicworks.com/service-areas/oc-infrastructure-programs/documents-maps/online-manuals

County of San Diego Department of Public Works Flood Control Section. (2005). *Drainage Design Manual*. http://www.sandiegocounty.gov/dpw/floodcontrol/floodcontrolpdf/drainage-designmanual05.pdf

Hoppe, E. J. (2011). Evaluation of Polypropylene Drainage Pipe. Virginia Center for Transportation Innovation and Research.

Hotii, R. S. (1988). *Bureau of Engineering Manual: Part H Structural Design*. City of Los Angeles Bureau of Engineering. http://eng2.lacity.org/techdocs/str-man/h-200.pdf

Los Angeles County Flood Control District. (1982). *Design Manual Hydraulic*.

Los Angeles County Flood Control District. (1982). *Design Manual Structural*.

Public Works Standards, Inc. (2009). *Standard Plan Public Works Construction (2009 Edition, Standard Plan 383-2)*. NBi Building News.

Soil Conservation Service. (1954). *Handbook of Channel Design for Soil and Water Conservation*. http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelprdb1043100

# Chapter 5 Culverts

## 5.1 Introduction

This chapter discusses the design criteria, hydraulic design, inlet, and outlet design for construction of culverts within the County of Orange. A culvert is considered a conduit, usually placed under a fill, such as a road or freeway embankment, to convey surface flow from the upstream side to the downstream side. Culvert design includes hydraulic design, proper location and alignment, debris loading, channel stability and sediment movement, minimization of long-term maintenance requirements, outlet channel protection and safety, structural, economic and life-cycle costs.

This manual provides only a basic level of information on culvert design criteria and design procedures. The Federal Highway Administration's *Hydraulic Design of Highway Culverts* (Schall et al., 2012) provides further information on culvert design.

## 5.2 Design Criteria

### 5.2.1 Protection Levels

Culverts herein are defined as hydraulic conveyances through the street embankment sections. Culverts shall be designed to discharge a minimum of 10-year storm without static head at the entrance, and with sufficient freeboard to discharge a 100-year storm. Storm flows shall be based on a bulked flow condition. Consideration must be given to:

1. 100-year protection to structures or top of curb elevation, whichever is lower
2. Ponding and overflow damages to adjacent property or to the roadway structure
3. Drainage system damages to the channel due to erosion and to the conduit due to scour or siltation
4. Definition Sketch

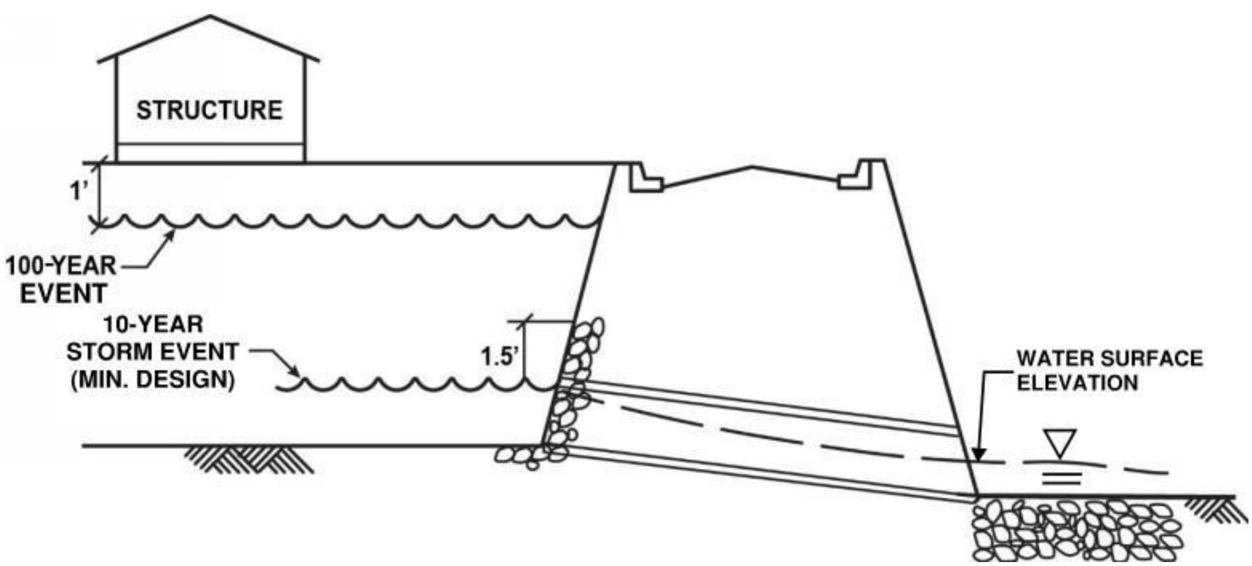



**Figure 5-1: Definition Sketch for Culverts**

### 5.2.2 Hydraulic Criteria

This manual generally references its culvert design criteria and procedures to circular and rectangular cross-sections. These criteria and procedures can be adapted to equivalent cross-sections (e.g., arches and other non-circular or non-rectangular shapes) with due care. Culverts with non-standard cross-section shapes must also adhere to the design criteria in this manual. Multiple barrel culverts are acceptable, so long as each barrel meets minimum gradient, velocity criteria, and sizing.

The friction slope represents the rate of loss of head in the culvert barrel due to friction. The head loss (ft) is computed as $h_l = S_f x\ L$, where $S_f$ is the friction slope and L is the length of the barrel in feet. This head loss shall be computed when the control is at the outlet since head loss must be deducted from the total head available to cause flow. Additional information on the hydraulic design procedure is also in Section 4.3.1.

Culverts shall be designed to convey the bulked peak 100-year design flow for public roads. Whenever practical, the culvert shall maintain a minimum gradient of 0.5 percent, or a flow velocity of 4 fps when flowing one-quarter full. When outlet velocities exceed permissible velocities for the outlet channel, suitable outlet protection (e.g., energy dissipation or channel lining) shall be provided (see Chapter 8).

For culvert facilities within the public R/W, the minimum culvert size shall be a 24-inch diameter round pipe. Minimum pipe size shall be 36-inches for debris producing watersheds within Categories 3 or 4 per the latest edition of the Orange County Flood Control District Design Manual. Culverts with over 20-feet of fill shall have a minimum pipe size of 60-inch diameter. Culvert design should maintain a Froude Number less than or equal to 0.9 or greater than 1.2.

Culvert headwater elevations shall maintain a freeboard of at least one foot below the roadway top of curb and the finished floors of structures within the zone influenced by the culvert headwater. When a culvert crossing increases the existing limits of flooding, the project owner shall obtain a flowage easement from affected property owners as required by the governing agency. A culvert headwall or other slope protection is required.

Culverts shall follow the alignment and grade of the channel it connects to whenever possible, unless otherwise allowed for by OCPW. Staggered entrances or exits to culverts shall not be allowed unless otherwise approved by OCPW.

In multi-barrel boxes, windows in interior walls may be required to equalize flows. The need and design shall be evaluated on a case-by-case basis.

Capacity nomograms and charts prepared by the Federal Highway Administration may be used for the solution of culvert flow problems. The data were derived from scaled and full-size models and are presented in a form which greatly simplifies the task of determining conduit type and size for a given condition. This manual contains charts for the normal installations; however, for culverts that are not included, refer to the complete set of Federal Highway Administration capacity charts.

### 5.2.3 Special Culvert Considerations

#### 5.2.3.1 Pipe Material

A wide variety of materials may be used for construction of a drainage system, including reinforced concrete pipe, cast-in-place concrete conduit, corrugated steel pipe, and other materials. Where field conditions dictate the use of one pipe material in preference to others (e.g., corrosive soil conditions, presence of a groundwater table, fire prone areas, vandalism, or a seawater outfall), the reasons shall be clearly presented in the plans and specifications and shall be approved by OCPW.

#### 5.2.3.2 Use of Available Head

It is not always economical or practical to use all the available head. This applies, in particular, to situations where debris must pass through the culvert, where a headwater cannot be tolerated, or where the natural gradient is steep and high outlet velocities are objectionable.

#### 5.2.3.3 Debris Considerations

If debris is not retained upstream of the culvert by a properly designed debris basin or debris capture system, culverts shall be sized to pass debris through the culvert (See Chapter 9, "Debris Facilities").

If debris is passed through the culvert, the available head must be used to maintain or accelerate the velocity of the flow approaching the culvert instead of creating a pond at the entrance, which invites a blocked culvert. In areas with steep gradients, full use of the available head may develop excessive velocity resulting in abrasion of the culvert itself or downstream scour. A larger culvert operating with less velocity may be more economical than an energy dissipator.

#### 5.2.3.4 Pipe Abrasion

Pipe abrasion considerations can be found in Chapter 4, Section 4.2.12 "Pipe Abrasion"

#### 5.2.3.5 Tailwater Depth

The tailwater depth (TW) is the depth of flow in the downstream channel just downstream of the culvert outlet. It must be calculated for each culvert design regardless of class or type of operation.

### 5.2.4 Design Considerations

All culverts shall consider the following hydraulic and maintenance factors:

#### 5.2.4.1 Hydraulic Factors

Hydraulic factors include the following:

- Design discharge
- Shape and cross-sectional area of channel
- Velocity of approach (where there is no static head)
- Headwater elevation above invert at inlet
- Tailwater elevation above invert at outlet
- Slope
- Roughness
- Length
- Inlet, outlet conditions and related losses (shape of lip, wing walls, etc.)
- Bulking
- Debris

#### 5.2.4.2 Maintenance Factors

Maintenance factors include the following:

- Accessibility: Open channels and storm drains shall be made accessible for normal maintenance equipment for storm repairs. Access for cleaning/maintenance shall be provided at the upstream channel.
- Access ramps should be provided to culvert inlet and outlet inverts where:

- There is a high potential for inlet clogging requiring emergency access to unplug the inlet.
- Trash, sediment and/or debris may have a potential to accumulate requiring maintenance including:
  - Access to remove sediment and debris from upstream debris posts or sediment basins,
  - Watersheds with a high wildfire/burn potential that may generate excessive sediment and debris, and/or
  - Potential impacts from homeless occupation.
- There is an outlet dissipator structure that requires periodic maintenance and/or repair.
- Culvert flow velocities have a high potential for downstream scour requiring periodic maintenance to:
  - Prevent head-cutting under the downstream apron or culvert structure,
  - Prevent creation of a downstream scour pool/pond, and/or
  - To allow maintenance of downstream revetment.
- Practical maintenance of invert is beyond the reach of OCPW, O&M owned heavy equipment (invert or culvert flow line lower than 25' from the roadway surface).
- Maintenance of the culvert and/or invert from the roadway surface/shoulder would be considered impractical if impacted by:
  - High speed traffic,
  - Elaborate safety railings or railings higher than 3.5',
  - Traffic or truck load restrictions,
  - Narrow roadways less than 40' in width,
  - Lane and/or road closures are not practical, etc.
- There is a recreational trail element that has a need to connect to the roadway.
- There is a connected or adjacent BMP or mitigation element that may require maintenance access from the roadway.

- Access ramps provided for maintenance and upkeep of culverts and their ancillary features shall:
  - Be a minimum of 12 feet in width and have a minimum 10-foot (heavy broom finish) paved concrete surface with a 2 ft clear area, all weather surface on the shoulder.
  - Access ramp and paving shall be designed for an H20-44 truck load, but at a minimum shall be no less than 4" PCC with a minimum of #4 bars at 12" on center and over a 6" aggregate base.
  - Access ramp shall have a grade no steeper than 10% with a 2% cross fall towards the drainage area.
  - At the bottom of the ramp, a concrete pad with a 35-foot radius or 50' x 50' turn around area shall be provided to allow heavy equipment the ability to turn around and exit the facility. The pad shall also be designed for an H20-44 truck load.
  - Where required, access from the roadway shall be restricted by a locked chain link gate and fencing conforming to OC Standard Plan 600-3-OC.

The following considerations are required for access ramps to culvert invert:

- A Geotechnical report addressing ramps and associated structures
- Ramps shall have a maximum 10% grade and should slope downstream. Ramps shall have a minimum of 2% cross-slope toward channel.
- Access ramp width, excluding walls and curbs, shall be a minimum of 12 feet.

- Ramps shall be located as close as possible to cross streets, but at least 50 feet from the street R/W. See addendum #4 section B.5 of the OCFCD Design Manual (County of Orange, 2000).
    - Ease of repairs: The system shall be located so maintenance may be accomplished with a minimum of inconvenience to the public.
    - Need for repetitive maintenance: No system shall be constructed that requires continuous maintenance. Systems that require excessive maintenance (as defined by OCPW) shall require an agreement.

## 5.3 Hydraulic Design of Culverts

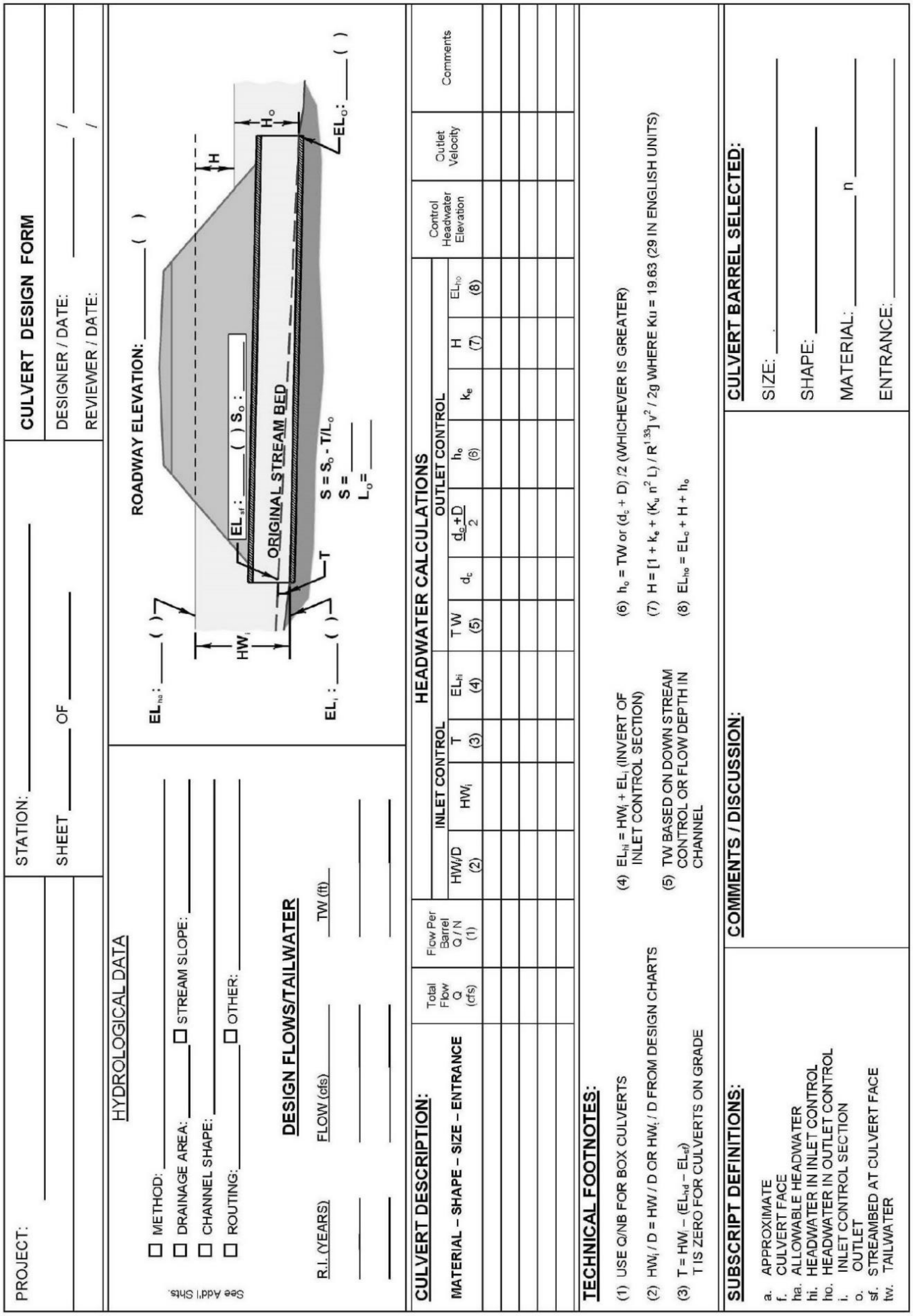
Culvert design is generally an iterative process. A trial size and type culvert must first be chosen with the expected headwater elevation being computed for the culvert carrying the design discharge. There are two major types of culvert flow: inlet control or outlet control. Figure 5-2 provides a form to guide the user through the design process. Figure 5-3 pertains to Inlet Control which is discussed in Section 5.3.1. The headwater depth (HW) occurs slightly upstream of the culvert entrance.

The headwater depth is the vertical distance from the culvert invert at the entrance (full cross-section) to the energy line of the headwater pool (depth + velocity head). Water surface and energy line at the entrance are assumed to coincide.

$$HW = H - h_0 - L \times S_0$$

Where:

| | | |
|---|---|---|
| $H$ | = | head (ft) |
| $h_0$ | = | tailwater (under conditions shown here) (ft) |
| $L$ | = | length of culvert (ft) |
| $S_0$ | = | slope of barrel (ft/ft) |



**CULVERT DESIGN FORM**

PROJECT: ______ STATION: ______ SHEET ______ OF ______

DESIGNER / DATE: ______ / ______

REVIEWER / DATE: ______ / ______

See Add'l Shts.

**HYDROLOGICAL DATA**

☐ METHOD: ______ ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______ ☐ CHANNEL SHAPE: ______ ☐ ROUTING: ______ ☐ OTHER: ______

**DESIGN FLOWS/TAILWATER**

| R.I. (YEARS) | FLOW (cfs) | TW (ft) |
|---|---|---|
| | | |
| | | |
| | | |



**CULVERT DESCRIPTION:**

**HEADWATER CALCULATIONS**

| MATERIAL – SHAPE – SIZE – ENTRANCE | Total Flow Q (cfs) | Flow Per Barrel Q / N (1) | INLET CONTROL HW/D (2) | INLET CONTROL $HW_i$ | INLET CONTROL T (3) | $EL_{hi}$ (4) | TW (5) | OUTLET CONTROL $d_c$ | OUTLET CONTROL $\frac{d_c+D}{2}$ | OUTLET CONTROL $h_o$ (6) | OUTLET CONTROL $k_e$ | OUTLET CONTROL H (7) | OUTLET CONTROL $EL_{ho}$ (8) | Control Headwater Elevation | Outlet Velocity | Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |

**TECHNICAL FOOTNOTES:**

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i / D = HW / D$ OR $HW_i / D$ FROM DESIGN CHARTS

(3) $T = HW_i - (EL_{hd} - EL_{sf})$ T IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) $h_o = TW$ or $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = [1 + k_e + (K_u n^2 L) / R^{1.33}] v^2 / 2g$ WHERE $K_u$ = 19.63 (29 IN ENGLISH UNITS)

(8) $EL_{ho} = EL_o + H + h_o$

**SUBSCRIPT DEFINITIONS:**

a. APPROXIMATE
f. CULVERT FACE
ha. ALLOWABLE HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

**COMMENTS / DISCUSSION:**

**CULVERT BARREL SELECTED:**

SIZE: ______

SHAPE: ______

MATERIAL: ______ n ______

ENTRANCE: ______

**Figure 5-2: Culvert Design Form (Schall et al., 2012)**

### 5.3.1 Inlet Control

Inlet control depends on the cross-sectional area of the barrel, the inlet configuration or geometry, and the amount of headwater or ponding (Figure 5-3). It occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. In general, these kinds of culverts are short, steep, and have a free outlet. The control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream of the culvert entrance is supercritical. Shallow high-velocity flow characterizes culverts under inlet control.

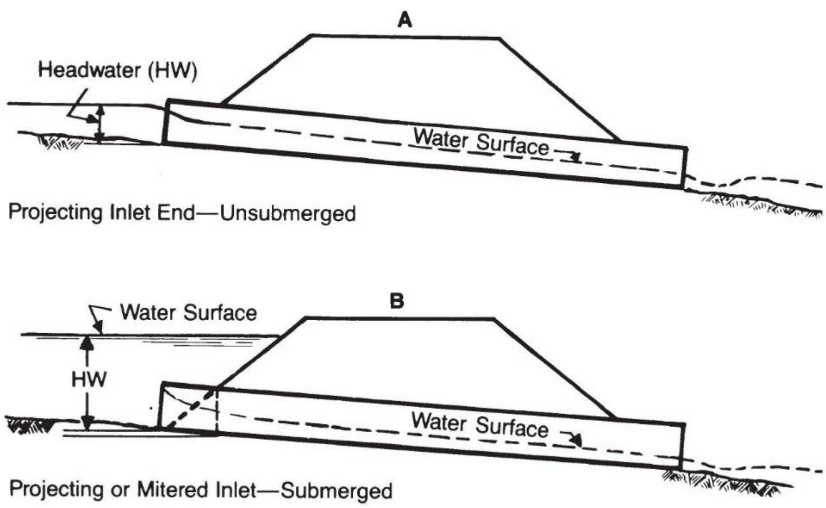



**Figure 5-3: Inlet Control (American Iron and Steel Institute, 1983)**

### 5.3.2 Outlet Control

Outlet control involves the additional consideration of the tail water in the outlet channel, and the slope, roughness, and length of barrel (Figure 5-4). It occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. In general, these kinds of culverts are long, rough-barreled with high tail water conditions. Either subcritical or pressure flow exists in the culvert barrel under these conditions. Therefore, the control section for outlet-controlled flow in a culvert is located at the barrel exit or further downstream.

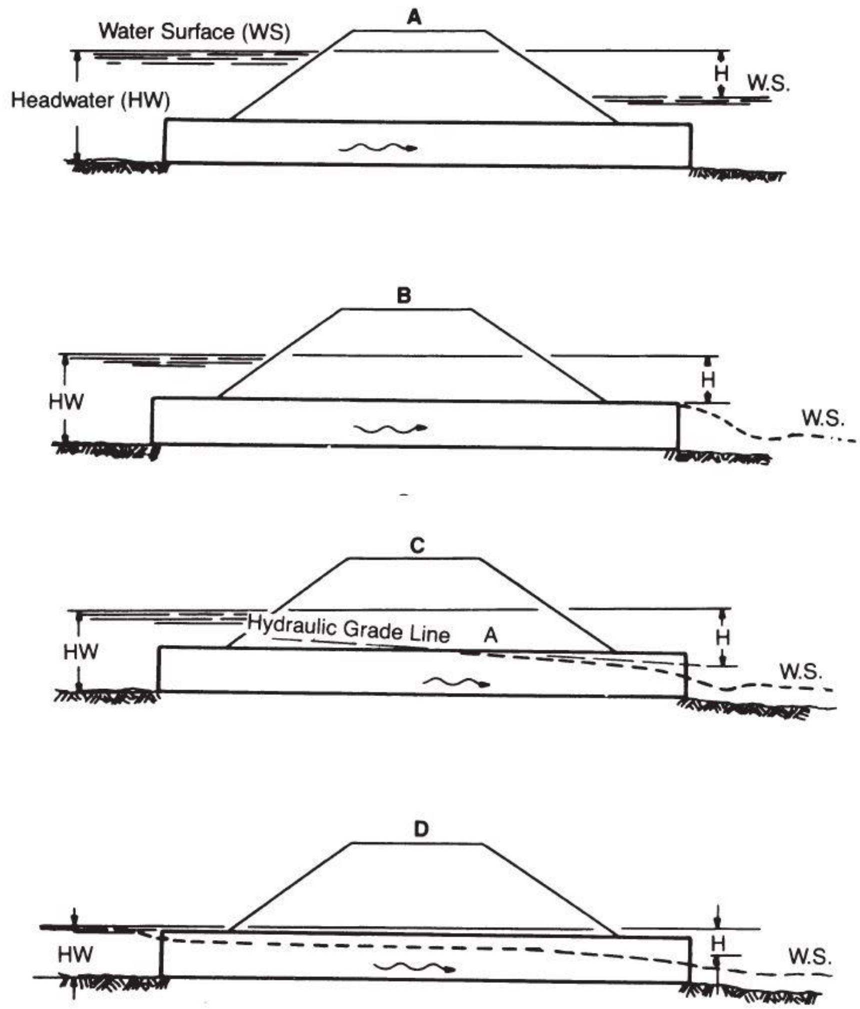



**Figure 5-4: Outlet Control (American Iron and Steel Institute, 1983)**

### 5.3.3 Selection of Design Condition

The culvert design condition is determined by comparing the calculated culvert headwater assuming inlet control with the calculated culvert headwater assuming outlet control. The condition with the higher culvert headwater elevation is the design condition.

### 5.3.4 Design Considerations

#### 5.3.4.1 Entrance Design

The design of an improved culvert entrance lowers the headwater pool, helps maintain or increase the velocity of approach, and may reduce conduit sizing. Entrance improvements shall consist of rounded-lip entrances, expanded entrances, or simply headwalls. A coefficient, $K_{en}$ is multiplied by the velocity head to determine energy losses at the culvert inlet. The entrance coefficient, $K_{en}$, from Figure 5-5 should be used in conjunction with the following criteria for culvert design:

1. The rounded-lip type entrance shall be used on all culverts. The hydraulic advantage of a rounded lip can be lost if the headwall end of the wing wall is offset from the sides of the pipe;
2. The expanded entrance is a more efficient entrance (such as a belled or throated entrance) for pipes under 36 inches in diameter.
3. Transitions (training walls) are used to guide the flow smoothly from the channel to the culvert entrance. Wing walls in conjunction with headwalls and concrete aprons are also used for this purpose. In most cases, wing walls perform the dual function of a hydraulic transition structure and embankment-retention structure. Wing walls, which act as training walls, should be considered for high channel velocities, hydraulic advantage, or where flow currents approach the inlet at an angle.
4. The use and sizing of headwalls shall be governed by the potential erosion damage both on slopes being retained and on adjacent slopes and the amount of material to be retained. However, headwalls with vertical drops over 30 inches shall be provided with protective safety railing depending on the use. For headwalls only accessible to operations and maintenance personnel, the top of safety railing shall not be less than 42 inches in height and shall consist of chain link fencing or cable railing and as approved by OCPW. Where there is pedestrian and bicycle traffic, safety railing shall be designed per Caltrans Highway Design Manual, Chapter 200 and as approved by OCPW. Debris gates may be required in the upstream channel in park or high use areas creating potential for large debris accumulations.

| **Type of Structure and Design of Entrance** | **Coefficient, $k_{en}$** |
|---|---|
| **Concrete Pipe Projecting from Fill (no headwall):** | |
| Socket end of pipe | 0.2 |
| Square cut end of pipe | 0.5 |
| **Concrete Pipe with Headwall or Headwall and Wingwalls:** | |
| Socket end of pipe (grooved end) | 0.2 |
| Square cut end of pipe | 0.5 |
| Rounded entrance, with rounding radius = 1/12 of diameter | 0.2 |
| **Concrete Pipe:** | |
| Mitered to conform to fill slope | 0.7 |
| End section conformed to fill slope | 0.5 |
| Beveled edges, 33.7 or 45 degree bevels | 0.2 |
| Side slope tapered inlet | 0.2 |
| **Corrugated Metal Pipe or Pipe-Arch:** | |
| Projected from fill (no headwall) | 0.9 |
| Headwall or headwall and wingwalls square edge | 0.5 |
| Mitered to conform to fill slope | 0.7 |
| End section conformed to fill slope | 0.5 |
| Beveled edges, 33.7 or 45 degree bevels | 0.2 |
| Side slope tapered inlet | 0.2 |

| **Type of Structure and Design of Entrance** | **Coefficient, $k_{en}$** |
|---|---|
| **Headwall Parallel to Embankment (no wingwalls):** | |
| Square-edged on three edges | 0.5 |
| Three edges rounded to radius of 1/12 barrel dimension | 0.2 |
| **Wingwalls at 30 to 75 degrees to Barrel:** | |
| Square-edge at crown | 0.4 |
| Top corner rounded to radius of 1/12 barrel dimension | 0.2 |
| **Wingwalls at 10 to 25 degrees to Barrel:** | |
| Square-edge at crown | 0.5 |
| **Wingwalls parallel (extension of sides):** | |
| Square-edge at crown | 0.7 |
| Side or slope tapered inlet | 0.2 |

**Figure 5-5. Entrance Loss Coefficient, $K_{en}$, for Culverts (Brunner, 2016)**

#### 5.3.4.2 Outlet Design

Culvert outlets shall be designed to restore culvert discharge to natural flow conditions downstream. Outlets should be carefully scrutinized for conditions that produce scour. Where progressive scour is expected, corrective measures such as bank protection or transitions shall be considered.

Suitable outlet protection (e.g., riprap or channel lining) must be provided when outlet velocities exceed permissible velocities for the outlet channel lining material. Chapter 8, “Energy Dissipation”, discusses the various types and the design of energy dissipators.

When the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. The outlet velocity can be assumed the same as the velocity within the barrel at normal depth.

When the controlling headwater is under outlet control, and the outlet is not submerged, the depth of flow at the outlet shall be taken as the larger of either the tailwater depth or the critical depth in the culvert barrel. The outlet velocity may be determined by dividing the discharge rate by the cross-sectional flow area within the barrel, based on the chosen depth.

## 5.4 Culvert Types and Criteria

### 5.4.1 Reinforced Concrete Pipe (RCP)

Reinforced Concrete Pipe criteria can be found in Chapter 4, “Storm Drains”.

### 5.4.2 Corrugated Steel Pipe (CSP)

Corrugated Steel Pipe criteria can be found in Chapter 4, “Storm Drains”.

### 5.4.3 Reinforced Concrete Box (RCB)

#### 5.4.3.1 General

Reinforced concrete box conduits shall be designed per guidance in Chapter 10.

Concrete dimensions should be detailed horizontally or vertically on the profile, and parallel to or at right angles or radial to the centerline of conduit. Caltrans standard plans show details for single and multiple span reinforced concrete box culverts.

#### 5.4.3.2 Cover over Box

An essential aspect of RCB design is the height of fill/cover over the top of the box. This cover dissipates live loads from traffic. Structural Design for RCBs is provided in Chapter 10.

## 5.5 References

American Iron and Steel Institute. (1983). *Handbook of Steel Drainage & Highway Construction Products*.

Brunner, G.W. (2016). *HEC-RAS, River Analysis System Hydraulic Reference Manual (Version 5.0)*. U.S. Army Corps of Engineers.

California Department of Transportation. (2018). *Highway Design Manual: U.S. Customary Units (6th Edition)*.

County of Orange Public Facilities and Resources Department. (2000). *Orange County Flood Control District Design Manual.* *https://ocip.ocpublicworks.com/service-areas/oc-infrastructure-programs/documents-maps/online-manuals*

County of San Diego Department of Public Works Flood Control Section. (2005). *Drainage Design Manual*.

Hotii, R. S. (1988). *Bureau of Engineering Manual: Part H Structural Design*. City of Los Angeles Bureau of Engineering. http://eng2.lacity.org/techdocs/str-man/h-200.pdf

Schall, J. D., Thompson, P. L., Zerges, S. M., Kilgore, R. T., & Morris, J. L. (2012). *Hydraulic Design of Highway Culverts: Third Edition (HDS 5)*. Federal Highway Administration. FHWA-HIF-12-026, April.

# Chapter 6 Open Channels

## 6.1 Introduction

This chapter discusses the design criteria, hydraulic design, and channel lining types for construction of local drainage open channels in the County of Orange. The chapter concludes with a list of acceptable software applications for use in channel hydraulics modeling, and references.

This manual includes a general overview of small channel design, while the Orange County Flood Control District Design Manual includes an extensive discussion on open channel design. Open channels, with tributary areas less than 640 acres, for local drainage, are generally small channels owned by unincorporated County, Cities, or Homeowner's Associations. These channels include small roadside ditches, earthen or vegetated channels, rock riprap channels, and concrete channels that convey small local drainage areas (less than 640 acres) to a regional or sub regional stormwater conveyance facility. For larger catchment areas, the Orange County Flood Control District Design Manual contains extensive discussions on open channel design, and it should be used as a reference for items not covered in this manual. Tributary areas exceeding 640 acres shall be designed using the OCFCD Design Manual.

## 6.2 Design Criteria

The following design criteria shall be used for local drainage channels built for dedication to the County of Orange or private facilities within unincorporated Orange County. This manual does not supersede any information contained within the Orange County Drainage Area Management Plan (DAMP), and it is intended to be consistent with the DAMP.

### 6.2.1 Protection Levels

#### 6.2.1.1 Structures

The goal is to provide 100-year protection for all habitable structures pursuant to the Public Services and Facilities Element of the County's General Plan. Per Figure 3-1, the 100-year water surface elevation shall be a minimum of 1 foot below pad elevation in accordance with Orange County Ordinance No. 09-008. If a FEMA determined base flood elevation is provided on a Flood Insurance Rate Map, then the minimum 100-yr water surface elevation shall be a minimum of 1 foot below the lowest finished floor. Local channel facilities will provide 100-year protection to structures. Local channel facilities can be used in combination with street flow requirements to provide 100-year protection. The local channels discussed in this chapter can provide flood protection in many ways and therefore, may require different design criteria based on the channel's proposed use. The appropriate design storm event shall be reviewed and approved by OCPW.

#### 6.2.1.2 General Criteria

1. Open channels with tributary areas of less than 640 acres are to be designed for a minimum of 10-year frequency. Open channels draining a sump condition shall be designed to a 25-year frequency with secondary overflow or provide 100-year protection.
2. Channel design requirements for Regional or Sub-Regional facilities shall be in conformance with the OCFCD Design Manual and the Orange County Hydrology Manual and its subsequent addendums.

3. The Rational Method, as described in the Orange County Hydrology Manual and its subsequent addendums, shall be used to compute peak runoff for drainage areas less than 640 acres for sizing local channels.
4. Ultimate condition land uses shall be based upon General Plan Land Use or similar planned communities' assumptions shall be used to determine runoff loss rates.
5. Use of any hydrology study not prepared in accordance with the Orange County Hydrology Manual and its subsequent addendums shall not be accepted.

### 6.2.2 Hydraulic Design Criteria

#### 6.2.2.1 Freeboard

Freeboard is required to ensure that the desired degree of protection shall not be reduced by unaccounted factors that may affect facility hydraulics but that are not required to be specifically analyzed in design. These factors include, but are not limited to, variations in Manning's "n" due to channel bottom conditions, uncertainties in the selection of Manning's "n", variation in stage-discharge relationships, velocity variation from average velocity, sedimentation, debris, bulking, and air entrainment. If any of the above factors are expected to be significant, then its effect shall be separately estimated and necessary provisions shall be included in design to account for it. Refer to Appendix F for common definitions of freeboard.

For regional or sub-regional facilities and for supercritical channels see the OCFCD Design Manual. The following are minimum acceptable freeboards for local channel facilities.

1. Subcritical flow (Fr < 0.9) – 0.5 ft
2. Supercritical flow (Fr >1.2) – 1.0 ft
3. Channels that need to be designed with a Froude Number between 0.9 and 1.2, freeboard shall be added to the conjugate depth per consultation with OCPW.

### 6.2.3 Hydraulic Grade Line

A plot of hydraulic grade line (HGL), and a table of appropriate hydraulic design data ranges (minimum to maximum values) shall be provided on final file (signed) plans, including station, section, bed slope, Manning's "n", design flow, velocities, and frequency. For open channel flow, flow depth, critical depth, and Froude Number shall also be shown. A sample table is provided in Table 6-1.

| STA TO STA | HYDRAULIC DATA | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Q | W | Z | $S_o$ | Design Analysis (n=0.030) | | | Scour Analysis (n= 0.020) | | | $D_c$ | $V_c$ |
| | | | | | D | V | F | D | V | F | | |
| | cfs | ft | ft/ft | ft/ft | ft | fps | - | ft | fps | - | ft | fps |
| 66+10.85 To 73+00.00 | 100 | 5 | 2.0 | 0.005 | 2.0-2.50 | 4.1-5.2 | 0.6-0.7 | 1.75-2.0 | 5.1-6.5 | 0.8-1.0 | 1.81 | 7.8 |

**Table 6-1: Example Hydraulic Data Table**

Where calculated backwater at a street crossing raises upstream water surface more than 1' in a natural channel, floodplain easements shall be provided for in the design. 100-year level of protection for habitable structures shall not be replaced or superseded by these criteria.

### 6.2.4 Manning's Roughness Coefficient

Selection of an appropriate channel roughness value for a given channel section is important for potential scour, the hydraulic capacity analysis, and design of an open channel. The roughness value can vary significantly depending on the channel type and configuration, density and type of vegetation, depth of flows, and other hydraulic properties. Channels that consist of composite linings should use a horizontal variable roughness coefficient or a composite roughness coefficient depending on the software being used to calculate the hydraulics. Earthen and natural channels shall use a Manning's roughness coefficient of 0.020 to compute the potential for scour before establishment of vegetation.

| **Material/Conveyance Type** | **"n" Value** |
|---|---|
| Concrete Channels | |
| Vertical Walls | 0.014 |
| Trapezoidal | 0.015 |
| Engineered Earthen Channels | |
| Fine Silt and Sand (size determination) | 0.030 |
| Fine Silt and Sand (scour determination) | 0.020 |
| River Sand and Gravel | 0.025 |
| Coarse Gravel Mixed with boulders | 0.035 |
| Greenbelt Channels | |
| Maintained turf | 0.030 |
| Heavily weeded, no brush | 0.040 |
| Heavily weeded, moderate shrubs | 0.050 |
| Some weeds, heavy brush | 0.060 |
| Rock Slope Protection (Riprap) | 0.035 |
| Flush Grouted Riprap | 0.020 |
| Wire Revetment (Gabions) | 0.035 |
| Natural Streams | |
| Regular Section | 0.045 |
| Irregular Section | 0.060 |
| Mountain | 0.055 |
| Flood Plains | |
| Pasture or cultivated | 0.040 |
| Heavy weeds, light brush | 0.050 |
| Medium to dense brush | 0.090 |
| Willows | 0.170 |

**Table 6-2: Manning's Roughness Coefficient for Open Channels**

### 6.2.5 Maximum Permissible Velocity

Maximum permissible velocities based on the type of channel lining are shown in the following table. The maximum permissible velocity is defined as an average velocity at a cross-section. It is not an average velocity along a reach. The values listed here are a basic guideline. Higher design velocities may be used if appropriate technical documentation from the manufacturer is furnished and approved by OCPW.

| Material/Lining Type | Maximum Permissible Velocity (fps) |
|---|---|
| *Natural and Improved Unlined Channels* | |
| Sandy Loam, Noncolloidal | 1.7 |
| Silt Loam, Noncolloidal | 2.0 |
| Alluvial Silts, Noncolloidal | 2.0 |
| Ordinary Firm Loam | 2.5 |
| Volcanic Ash | 2.5 |
| Stiff Clay, Very Colloidal | 3.7 |
| Alluvial Silts, Colloidal | 3.7 |
| Shales and Hardpans | 6.0 |
| Fine Gravel | 2.5 |
| Graded Loam to Cobbles When Noncolloidal | 3.7 |
| Graded Silts to Cobbles When Colloidal | 4.0 |
| Coarse Gravel, Noncolloidal | 4.0 |
| Cobbles and Shingles | 5.0 |
| Sandy Silt | 2.0 |
| Silty Clay | 2.5 |
| Clay | 6.0 |
| Poor Sedimentary Rock | 10.0 |
| *Fully-Lined Channels* | |
| Unreinforced Vegetation | 5.0 |
| Reinforced Turf | 10.0 |
| Loose Riprap | |
| No. 2 Backing | 10.0 |
| ¼ ton (D50 = 23”] | 12.0 |
| ½ ton (D50 = 28”) | 14.0 |
| 1 ton (D50 = 36”) | 16.0 |
| 2 ton (D50 = 45”) | 18.0 |
| Special Design | >18.0 |
| Grouted Riprap | 20.0 |
| Gabions | 15.0 |
| Concrete | 20.0 |

**Table 6-3: Maximum Permissible Velocities for Channels (adapted from Caltrans, 2018, Table 865.2)**

### 6.2.6 Subcritical and Supercritical Flow

Local channels should generally be designed for subcritical flow regimes; however, if the design requires supercritical flow design, the requirements in the OCFCD Design Manual should be followed.

### 6.2.7 Channel Alignment

Should the channel require horizontal curves, the procedures for super elevation and wave action need to be addressed as discussed in the OCFCD Design Manual.

### 6.2.8 Access and Safety

Access to local channels depends on the type of channel. Some channels will require maintenance roads as described in the OCFCD Design Manual. Many of the smallest channels that are owned by private entities such as terrace drains and v-ditches have minimal access. Care must be taken to assure maintenance access for all channel types. Any facilities to be owned and maintained by the County of Orange require access to be approved by OCPW.

The requirements for OCPW maintenance include:

- For Channel base widths equal to or greater than 10 feet, a vehicle access road is required.
- For Channel base widths less than 10 feet, a 5-foot wide walkable access path is required

## 6.3 Hydraulic Design of Local Channels

### 6.3.1 Channel Analysis – Uniform Flow

An open channel can be evaluated numerically using the Uniform Flow Equation:

$$Q = \frac{1.486}{n} AR^{2/3} S_f^{1/2}$$

Where:

| | | |
|---|---|---|
| $Q$ | = | flow rate (cfs) |
| $n$ | = | Manning roughness coefficient (no dimension) |
| $A$ | = | flow area (ft$^2$) |
| $R$ | = | hydraulic radius (ft) |
| $S_f$ | = | friction slope (ft/ft), assumed equivalent to a channel's longitudinal slope ($S_\emptyset$) |

### 6.3.2 Channel Analysis – HGL Calculations

The designer shall check the available energy at all junctions and transitions to determine whether the water surface elevation in the channel will be impacted due to backwater effects. Methods such as the direct step method presented in Section 4.3 Hydraulic Design of Storm Drains can be used for open channels. The OCFCD Design Manual also discusses open channel hydraulic HGL calculations. Computer programs are generally used for channel HGL computations. A list of accepted software is included in Section 6.5.

### 6.3.3 Downstream Control (Tailwater) Elevation

The hydraulic analysis of a subcritical channel system typically begins at the downstream outfall. The controlling water surface elevation at the point of discharge is commonly referred to as the tailwater. At the outfall, one of several conditions will be encountered: a closed conduit; outfall to a drainage channel, storage facility, reservoir, lake, or detention facility; free outfall; or tidally influenced outfall.

The tailwater elevation criteria described here are for determining HGL and Energy (EGL) elevations only.

For free outfalls, the initial water surface elevation (tailwater) shall be assumed equivalent to the critical depth in the channel. Outfalls to other drainage facilities, such as a drainage channel, reservoir, or detention facility, the initial water surface elevation shall be set at the design flow water surface elevation calculated for the downstream facility. If the design flow of the receiving facility is less than a 25-year storm event, a new 25-year water surface elevation shall be calculated and used as the initial water surface elevation for the design of the new facility. If an event lower than the 100-year is used to design the system, the development must meet the 100-year flood protection requirements when evaluated using the 100-year water surface in the downstream channel.

If no downstream information is available, the water surface elevations identified on the appropriate Flood Insurance Rate Map (FIRM) at the location of the outfall may be used if approved by OCPW. In cases where the storm drain outfall condition is tidally influenced, it is usually sufficient to use the mean higher high-water elevation as the tailwater elevation. In cases where storm surge is a concern or for other situations with unusual tailwater conditions, the appropriate design outfall tailwater elevation shall be chosen in consultation with OCPW.

## 6.4 Types of Local Open Channels

There are many types of local open channels: roadside ditches, water quality bio swales, v-ditches, slope drains and channels. Local open channel linings include non-reinforced concrete, reinforced concrete, grouted rock riprap, loose rock riprap, earthen, and vegetated. All local channels can be classified as either rigid or flexible.

### 6.4.1 Rigid Channels

Local Rigid Channels can be either v-shaped, trapezoidal or rectangular. The local channel facility lining can be reinforced concrete, interlocking block, or grouted riprap. They are generally used in areas where there are space constraints, high velocities, and/or high abrasion and erosion potential. Rigid channels are usually more costly to construct than flexible channels, but they are less costly to maintain and have a longer service life.

#### 6.4.1.1 Reinforced Concrete Channels (RCC)

Reinforced Concrete Channels are either trapezoidal or rectangular channel sections. Therefore, they are useful in space-constrained locations. They can also be used in areas where infiltration is undesirable or to eliminate stagnant water. See Figure 6-1 for an example of a trapezoidal concrete lined channel.

**Figure 6-1: Concrete Lined Channel Example**

### 6.4.1.2 V-ditches and Slope Drains

V-ditches and slope drains are concrete channels with minimal reinforcing steel that are generally used to drain small areas (less than 1 acre) and are sized by normal depth calculations. OCPW Standard Plan Nos. 1321, 1332, and 1334 are all examples of these types of channels. Figure 6-2, Figure 6-3, and Figure 6-4 show the different facilities.

**Figure 6-2: Terrace Drain Example**

**Figure 6-3: Down Drain Example**

**Figure 6-4: V-ditch Example**

#### 6.4.1.3 Interlocking Block Channels

These types of channels are lined with prefabricated products such as concrete block or articulated concrete block. The concrete block lining is used in areas where aesthetics are important. They can be used as slope lining with natural channel inverts and grade control structures. Articulated concrete block is concrete block that may or may not be connected by cables to form concrete block mats. Articulated concrete block lining may provide soil voids for vegetative growth. Design and construction of articulated concrete block channels requires diligence to assure that the product performs in flow conditions as anticipated. Block materials may be less expensive than concrete if local sources are available. A soils report which addresses the device used is required. Interlocking block structures are considered temporary (non 100-year) structures.

#### 6.4.1.4 Grouted Rock Riprap Transitions

Grouted rock riprap channels are rock lined channels with concrete added to fill the surface rock voids. This initially provides a rigid channel lining but may become an obstacle to aquatic organism passage if a dual invert develops due to local scour. Grouted rock riprap channels are generally disallowed without special design considerations. They are often used as transitions from reinforced concrete structures to flexible lined channels because the increased roughness of the rock helps dissipate energy before transitioning to flexible lining. Grouted rock riprap may also be used to repair channel lining failure areas. Grouted rock structures are considered temporary (non 100-year) structures.

### 6.4.2 Flexible Channels

Flexible linings such as earthen or vegetated channels are generally trapezoidal channels with mild side slopes of 3:1 or flatter such as roadside ditches, water quality vegetated swales, and channels. They are generally used in areas with uniform flow and lower velocities. Channel lining includes vegetation, rock riprap, gabions, and turf reinforcing mats. Flexible linings are good for environmental considerations such as water quality, habitat, environmental permitting, and green street design. Flexible channels tend to have a lower initial construction cost, but higher long-term maintenance costs with a shorter service life than rigid linings. Riprap may be used with steeper side slopes and higher velocities.

#### 6.4.2.1 Rock Riprap Channels

Rock riprap channels are loose rock lined channels. The stones used for channel lining are generally large diameter angular rock. Loose rock lined channels require a filter material between the stones and the underlying soil to prevent migration of foundation soils. Riprap lined channels are capable of withstanding higher velocities than vegetated channels and are often used in combination with other rigid or flexible linings. A “Toe Structure” is required to hold the up-slope rip-rap in place. Loose rock riprap lined channels may be less expensive than grouted rock lined channels and can adapt their shape to any underlying soil loss, unlike grouted rock. Figure 6-5 shows rock riprap lining on the sides of the earthen channel.

**Figure 6-5: Rock Riprap Channel Example**

### 6.4.2.2 Gabions

Gabions are rock-filled wire enclosed baskets or mattresses. The gabions can be used either as channel lining or as grade stabilization structures in a channel. Using gabions as grade stabilization can flatten slopes in a channel to lower velocities. These types of structures are useful in steep channels that require grade control. Caltrans Standard Plan D100A and D100B may be used as a reference for gabion structures. The costs to design and construct these types of channels are relatively expensive; however, they may be the only option for resource agency requirements. Figure 6-6 shows a gabion-lined channel. Gabions are considered temporary (non 100-year) structures.

**Figure 6-6: Gabion Channel Lining Example**

#### 6.4.2.3 Vegetated Channels

Vegetated channels include various types of channel lining such as grass, willow cuttings, and wetland plants. Local vegetated channels in Orange County are typically used for small tributary areas. Typical uses are roadside water quality vegetated swales as part of a green streets project or a project's NPDES compliance. Vegetated channels are very susceptible to high channel velocities. They can only be used in channels with a subcritical flow regime and a non-erosion invert. They are generally inexpensive for initial construction but require more maintenance such as trimming or thinning of vegetation, than rigid linings. These types of channels are generally preferred by resource agencies because they can provide water quality and habitat benefits, but proper design can be very complex. Figure 6-7 shows a vegetated channel.

**Figure 6-7: Vegetated Channel Example**

#### 6.4.2.3.1 Roadside Ditches

Earthen or vegetated roadside ditches should be designed in accordance with FHWA HEC-15 procedures for Vegetative Lining and Bare Soil Design. Paved ditches shall be designed in accordance with Caltrans Highway Design Manual Section 834.3. Key variables to consider are the susceptibility of the vegetation to lay down during flow events and the cohesiveness of the soil being used. Generally, roadside ditches can be designed using normal depth procedures if the ditch is flowing in a subcritical flow regime. Figure 6-8 shows a roadside ditch.

**Figure 6-8: Roadside Ditch Example**

#### 6.4.2.3.2 Water Quality Vegetated Swales

In Orange County, water quality vegetated swales are designed in accordance with the Water Quality Management Plan (WQMP) Technical Guidance Document (TGD). The vegetated swale design constraints are detailed in the TGD BMP Factsheet Appendix XIV BIO-2 Vegetated Swale. The 2013 version of the TGD states the following about vegetated swales: Tributary areas should be less than or equal to 5 acres; the overall slope of the site should be less than 10 percent; the maximum bed slope should not be more than 6 percent, unless check dams are used; Velocities in water quality vegetated swales should be less than 1 fps; the swale must be at least 100 feet long. Figure 6-9 shows a water quality vegetated swale.

**Figure 6-9: Water Quality Vegetated Swale Example**

### 6.4.2.4 Turf Reinforcing Mat Channels

Turf reinforcing mat channels are non-biodegradable rolled or woven erosion control products that are made of UV resistant synthetic fibers. They are used in combination with vegetated channels to improve vegetation erosion resistance. Turf reinforced channels are less expensive than rock lined channels and can facilitate using vegetation in velocities that would usually require rock lining, which makes them a preferred channel lining material with resource agencies.

## 6.5 Acceptable Software

The following are acceptable software for open channel hydraulic calculations:

- Bentley Hydraulics
- Federal Highway Administration (FHWA) Hydraulic Toolbox
- Advanced Engineering Software (AES)
- AutoCAD Hydraulic Software
- Hydraulic Engineering Center River Analysis System (HEC-RAS)
- Water Surface Pressure Gradient for Windows (WSPGW)

If a computer application other than those listed above is used, documentation needs to be provided to OCPW with justification for its use. Additional review and permitting fees may be required for the County to hire specialized expertise to perform an adequate review.

## 6.6 References

County of Orange Public Facilities and Resources Department. (2000). *Orange County Flood Control District Design Manual*. http://ocflood.com/docs/manuals

County of San Diego Department of Public Works Flood Control Section. (2014). *San Diego County Hydraulic Design Manual*. https://www.sandiegocounty.gov/content/dam/sdc/dpw/FLOOD_CONTROL/floodcontroldocuments/hydraulic_design_manual_2014.pdf

Kilgore, R. T., & Cotton, G. K. (2005). *Design of Roadside Channels with Flexible Lining*: *Hydraulic Engineering Circular No. 15 (HEC 15), Third edition*. Federal Highway Administration. FHWA-NHI-05-114, September.

OC Watersheds. (n.d.). *Model Water Quality Management Plan (WQMP).* Retrieved June 1, 2020 from http://www.ocwatersheds.com/documents/wqmp

# Chapter 7 Local Detention Basins

## 7.1 Introduction

This chapter discusses the design criteria, standard features, maintenance criteria, water quality considerations, and design procedures for local detention basins within the County of Orange. The chapter concludes with discussion of acceptable software for use in local detention basin modeling and references.

## 7.2 Design Criteria

A local detention basin is defined as a basin with a tributary area of 640 acres or less and basin grading meeting the city or county jurisdictional requirements as shown in Figure 7-1. Local detention basins include debris basins, local peak flow and volume mitigation basins, and water quality basins. The following design criteria shall be used for local basins built for dedication to the County of Orange, cities, or private facilities within unincorporated Orange County.

Design storm frequencies are subject to individual review by OCPW and should be in accordance with the Orange County Hydrology Manual, its subsequent addendums, and this manual's Flood Protection Goals. This manual does not supersede any information contained within the Orange County Drainage Area Management Plan (DAMP). It is intended to be consistent with the DAMP.

### 7.2.1 Protection Levels

The protection goals of each type of local facility vary. However, all online basins shall function within a drainage system that provides 100-year storm event protection of all habitable structures pursuant to the Public Services and Facilities Element of the County's General Plan. Individual basins shall not be permitted to contravene this overall tract protection objective irrespective of a facility's design criteria. The varying general design criteria for each basin type are discussed below:

1. Debris Basins: The combined pipe and spillway outflow shall not exceed the downstream conveyance capacity and are to be designed per OCPW Standard Plan 1327. Debris facility design is covered in Chapter 9, "Debris Facilities," of this manual.
2. Local Detention Basins: These facilities are generally used to reduce peak flows and/or volumes to downstream facilities. The storm events required depend on the downstream facility. For example, a downstream natural channel may require mitigation for the 2-, 5-, 10-, 25-, 50-, and 100-year storm event. A basin tying into a downstream storm drain may be used to reduce peak flows to the downstream facility's design storm event. Regardless of the intended use, the basin will be designed to accommodate the inflow, outflow and storage of the 100-year high confidence storm event (see OCFCD Design Manual). Should the basin take more than 24-hours to drain, a multi-day analysis of the basin is required.
3. Water Quality Basins: The primary purpose for these basins is to improve quality of storm water runoff. They are generally designed to treat runoff volumes that are less than typical storm drain design events. Therefore, if the only function of the basin is for water quality, it should be designed as an off-line facility and follow the guidance in the OC DAMP Technical Guidance Document and per OC Watershed criteria.
4. Hydromodification Basins: Hydromodification basins are designed to mimic existing or pre-development flow duration, flowrate, volume, and time of concentration for small (generally 2-

year) storm events. These basins shall conform to the guidelines in the OC DAMP Technical Guidance Document.

#### 7.2.1.1 Design Complexity

The hydraulics of open channels is relatively simpler than the hydraulics of bare-earth retarding basins. The hydraulics design complexity increases as the designer aims to associate the basin with a channel system for routing purposes. Low Impact Developments (LIDs), hydromodification, and water quality (WQ) are interrelated design topics. Introduction of these design topics into the hydraulics, of conjunctive use basins, increases the level of design complexity beyond the scope of the LDM.

### 7.2.2 Jurisdictional Dams

The State of California defines a dam as any artificial barrier, together with appurtenant works, that impounds or diverts water (California Water Code, Division 3, Section 6002-6003). The State Division of Safety of Dams (DSOD) regulates basin structures that are:

1. Over 6 feet high with 50 ac-ft of storage, or
2. Over 25 feet high with 15 ac-ft of storage (See Figure 7-1) per Section 6307(b) of the California Water Code

DSOD determines the jurisdictional height of a dam. The dam height is defined by the California Department of Water Resources (CA-DWR) as “the vertical distance measured from the lowest point at the downstream toe of the dam to its maximum storage elevation, which is typically the spillway crest.” (CA-DWR, Jurisdictional Sized Dams).

Here, height of a dam is further clarified as the vertical distance (ft), above the natural bed of the stream or watercourse at the downstream toe of the dam barrier. Alternatively, the height may be measured from the lowest elevation of the outside limits of the dam barrier to the maximum possible water storage elevation. The final decision on the jurisdictional height of the dam is per DSOD’s determination.

Design of Jurisdictional Dams is not covered in this manual. The design of these facilities is governed by DSOD and should meet its design standards.

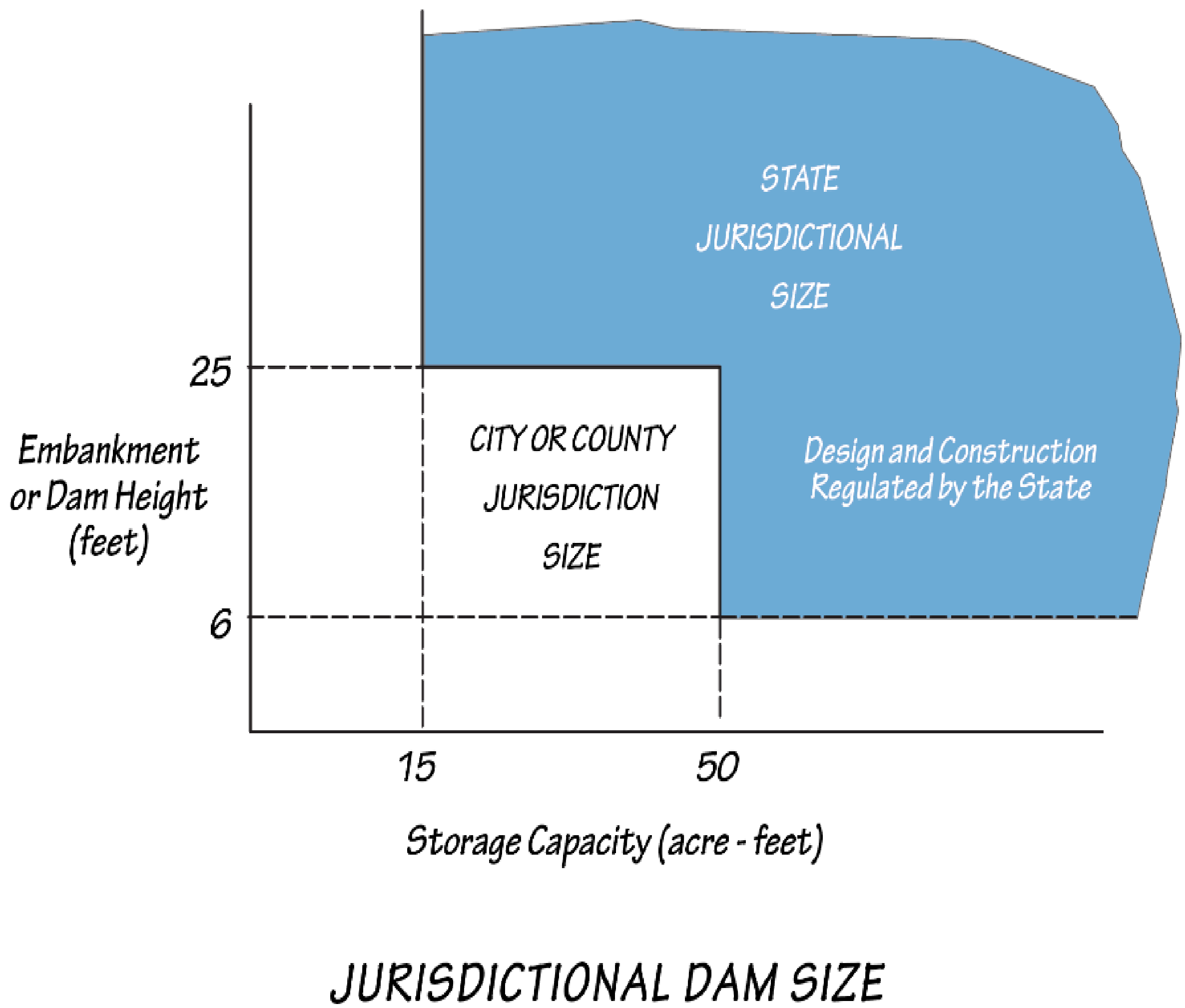



**Figure 7-1: Jurisdictional Dam Diagram (Orange County Public Works, 2018)**

### 7.2.3 Grading and Embankment Slopes

For local basins, both upstream and downstream slopes can be designed with slopes based on geotechnical and structural analyses to support the design. Embankment soil must be compacted. The in-place density/relative compaction shall be 90% in conformance with the requirements of OCPW Standard Plans 1322, 1323, and 1801. The entire area to be occupied by the embankment shall be stripped to a sufficient depth to remove all unsuitable materials, including surface boulders, loose rock, debris, topsoil, and vegetation, that might interfere with the proper bonding of the embankment with the foundation.

### 7.2.4 Standard Features (Inlets, Outlets, and Emergency Spillways)

This section provides the minimum design criteria for typical detention basin facilities including inlets, outlets, and emergency spillways.

#### 7.2.4.1 Inlets

Detention facilities shall have adequate energy dissipation and/or erosion protection at the facility inlet to avoid damage as flow enters the facility. Chapter 8 provides a discussion of energy dissipation devices. Incorporating Forebays and sediment traps at inflow points to larger basins can reduce the amount of sediment and debris to the main part of the facility and are required unless otherwise approved by OCPW.

#### 7.2.4.2 Outlets

Outlet structures must be carefully designed to ensure proper facility operation, to facilitate maintenance, and to maintain safety. Outlet structures for detention facilities shall be designed to safely convey the design release rate as discussed in Section 7.2.1.

Outlet structures shall be constructed with no moving parts whenever practical. Anti-seepage design shall be provided for all outlet structures conveying flow through a detention basin embankment. Appropriate energy dissipation shall be provided downstream of all detention facility outlet works (see Chapter 8).

Riser pipes and culvert outlet structures shall be equipped with debris racks, screens, or anti-vortex devices in order to help prevent clogging, and to prevent entry by unauthorized persons. These appurtenances shall be well secured but removable for the purposes of maintenance. Debris racks must not interfere with the hydraulic capacity of the outlet.

#### 7.2.4.3 Emergency Spillways

Emergency spillways provide a safe means for conveying flows in excess of the maximum design capacity of the outlet works. Spillways shall be designed to pass flow from an uncontrolled 100-year high confidence design event (i.e., the maximum 100-year peak flow that enters the basin) without overtopping the basin at a minimum. The design engineer shall evaluate the risk associated with uncontrolled discharge from the basin and design the emergency flow path accordingly. Spillways shall be appropriately protected to prevent excessive damage to the structure and adjacent property during spill events.

### 7.2.5 Detention Facility Plans

The design engineer shall note the following on detention facility drawings: maximum design inflow and velocity; maximum total design outflow and velocity from the outlet works; stage-storage and stage-discharge curves including the maximum design storage volume and water surface elevation in the facility; and the maximum design flow, depth, and velocity over the emergency spillway. Plans for detention facilities shall include appropriate details for the facility inlet, outlet structures, energy dissipators, emergency spillway, maintenance measures, and cross-sections of embankment fills.
The “Plans” shall be prepared on separate sheets with the agency responsible for maintenance noted on “Plans”.

### 7.2.6 Maintenance Criteria

All detention facilities require maintenance to ensure proper function throughout their lifetime. This section describes the minimum requirements for maintenance of detention facilities, including maintenance access, appropriate easements and environmental permitting, and plans for operation and maintenance. Additional maintenance requirements may be required on a case-by-case basis.

#### 7.2.6.1 Operation and Maintenance Plan

An operation and maintenance manual describing all features of basin operation, including a copy of all regulatory permits and conditions (construction and post-construction operation and maintenance), basin agreements, rating curves of all facilities, stage-storage/stage-discharge curves for the basin, instruction manuals and warrantees for all equipment (especially pump stations), basin facility record drawings (as-built plans) on 11x17 sheets, and a maintenance schedule shall be provided. Basin Facility record drawings (as-built) Plans in electronic format (PDF) or on 11x17 inch sheets, if specifically requested, shall be included. A maintenance schedule shall be provided on the Plans.

A separate operation and maintenance manual shall be required for pump stations, automatic gate facilities, or any other machinery or automatic operating features within the basin.

A basin operation and maintenance manual shall at a minimum contain the following:

1. The manual shall include a vicinity and location map in a chapter dedicated for site access. This chapter shall include any special instructions, directions, and specific details for ingress and egress of the facility during full and non-operating situations.

2. A second chapter in the manual shall specifically address any regulatory permits and agreements for basin operation and maintenance. General and/or special conditions that impose specific restrictions or require action on the part of basin maintenance and operation personnel shall also be listed.

3. Complete and detailed descriptions and instructions for the basin facility during nonoperation, full operation and emergency situations, including how to operate any gates, valves, and other operating systems required for complete operation of the facility.

4. Each routine maintenance activity shall list the activity and the frequency of required inspection. Routine maintenance activities include:
   - Sediment removal
   - Debris and trash removal
   - Vegetation control
   - Vector control (rodents, mosquitoes, etc.)
   - General maintenance and repair items, such as corrosion control of metal components
   - O&M of any structural BMP components

5. A summary of inspection items for each inspection activity (pre, post, and during storm season). See Appendix C for an example of a basin inspection items summary prepared for an existing basin facility.

6. Detailed schedule and description of inspections of all basin facilities:

   - <u>Pre-storm Season Inspections:</u> The purpose of the pre-storm season inspections is to confirm the readiness of the drainage facilities to accept storm runoff, to identify deficiencies or possible facility deficiencies, and to perform or schedule maintenance to prepare the facilities for the storm season.

   - <u>Storm Season Inspections:</u> The purpose of the storm season inspections is to clear trash, debris, sediment or other obstructions from drainage ditches, terrace drains, inlet, and outlet structures that could cause flow diversions, erosion, and displacement of basin materials. The storm season inspections are also conducted to identify failures, or potential failures of basin components, and to schedule emergency corrective action, if so warranted.

- Post-Storm Season Inspections: The purpose of the post-storm season inspections is to conduct a thorough accounting of all basin structures, components, and appurtenant items for damage, wear and tear, and potential/future failure. After such inspections, routine maintenance should be conducted, and operational repairs scheduled and implemented accordingly.

7. Facilities for Inspection should include:

- Surface Drainage Facilities: Embankments, slopes, surface drainage facilities including terrace and drainage ditches and culverts, all gates and gate operating systems, flap gates, hinges, joints, manholes, inlets, outlets, spillways, riprap, dissipators, intake structures, splash walls and pads, baffle blocks, debris basins, debris deflectors, debris posts, etc.

  Surface drainage facilities shall be inspected for structural damage, exposed reinforcing steel, heavily abraded or eroded concrete, transverse cracks, sulfate damage to concrete, misaligned joints, undermining due to longitudinal erosion, obstruction due to slope sloughing, sediment deposition or debris accumulation, settlement, obstructed weep holes or intake tower slots, displaced riprap, degraded rock revetment or facing materials, voids, unprotected subgrade soils, exposed or damaged geo-fabric, damaged, plugged, or missing trash racks/grates, coating (paint) degradation or corrosion of metal components, and vandalism or graffiti.

- Subsurface Drainage Facilities: Subsurface drainage facilities include all conduits that pass through the basin embankments, subgrade drainage systems (if any), etc. Subsurface Drainage Facilities shall be inspected for misaligned joints, crushed or cracked conduits, seepage through joints, piping of fines along conduits extending through or within the basin embankments or slopes, joint separation, settlement, vegetation growing through joints, debris deposition, coating degradation or corrosion of metal components, damaged or plugged inlets, and damaged manhole or access way components.

- Access Roadways and Access Ramps: Access roadways and ramps shall be inspected for settlement or upheaval of paving materials, excessive cracking, need for seal coat for asphalt paving, damaged pavement edges, damaged curbs, gutters, or roadside drainage ditches, eroded or rutted crushed miscellaneous base surfacing, roadway wash-boarding and rivulets, damaged or missing site markers, and improper surface drainage.

- Graded Earthen Slopes: Earthen slopes shall be inspected for undesired vegetation, overgrowth, damage to or undermining of terrace drainage systems, damage to erosion control blankets, slope erosion, excessive rivulets creation, sloughing, loss of slope erosion cover, and slope settlement or bulging.

- Fencing and Gates: Fencing and Gates shall be inspected for bent or missing posts or top rails, damaged or missing fence fabric, debris accumulation against fencing, loose tensioners, bent or misaligned gates, inoperable latching hardware, and missing locks and chains.

- <u>Vegetation Control:</u> Hydroseeded areas of a basin project shall be inspected for spread to non-hydroseeded areas, growth within paved of other facility areas, and vegetative growth of undesired species.

- <u>Sediment and Debris Facilities:</u> Sediment/debris facilities include basins, ponds, trash racks, debris posts, etc. Sediment and debris facilities shall be inspected for debris pool elevation, sediment basin or retarding basin invert sediment accumulation elevations, plugged or missing trash racks, debris accumulation on debris posts, and damaged or loose debris posts.

### 7.2.6.2 Maintenance Access

Maintenance access may vary depending on the size and location of a particular basin. Unless otherwise waived by the OCPW Engineer, the following maintenance access requirements shall apply:

- An asphalt or concrete paved berm and/or maintenance road shall be provided around the entire periphery of the basin (except as noted below) which shall otherwise conform to the requirements of the OCFCD Design Manual for Public Works Maintenance Requirements. The maintenance and access roadway shall be graded to drain at a 2 percent (2%) cross slope toward a parallel asphalt or concrete berm or concrete drainage ditch (or “v”-ditch). The surface runoff shall be directed to a concrete down drain, storm drain, or dissipator device to effectively minimize any potential sheet flow erosion of the basin slopes from the maintenance/access roadway.
- At least one continuous 15’ wide, 6” thick, concrete paved access ramp (capable of withstanding H20-44 truck loading) with a maximum slope of 10 percent (10%) shall be provided from the nearest street to the invert of the basin. The access ramp shall be graded to drain at a 2% cross slope toward a parallel asphalt or concrete berm or concrete drainage ditch or “v”-ditch. The surface runoff shall be directed to a concrete down drain, storm drain, or dissipator device to effectively minimize any potential sheet flow erosion of the basin slopes.
- A concrete paved access ramp shall be extended to all basin outlets, intake towers, and other debris/sediment trapping appurtenances, where applicable. A 35’ radius, 6” thick, circular concrete pad (capable of withstanding H20-44 truck loading) shall be constructed around the base of all intake towers with a maximum cross slope of 2 percent (2%). A 50’ x 50’ wide or 35’ radius concrete turn-a-round shall be constructed at or near the terminus of the access ramp on the basin invert but shall be located 3’ above the maximum allowable basin sediment storage elevation. The requirement for concrete on the basin invert may be substituted for a more “Eco-Friendly” material with the approval of OCPW.
- All basin access ramps, turn-a-rounds, and outlet pads shall have a heavy broom finish for maintenance and emergency vehicle tire traction during severe weather conditions.
- The maintenance road entrance to the basin shall not be in the direct path of emergency spillway flows unless an alternate route is provided to access the basin property and appurtenances. In cases where a weir or spillway interrupts the continuity of the berm and maintenance road, the roadway shall provide a 50’ x 50’ turn-a-round or a cul-de-sac with a 35’ radius turn-a-round at each side of the weir/spillway, or the maintenance road shall bridge the weir/spillway.
- The maintenance/access roadway system shall be designed such that the entire periphery of the basin shall be accessible during a design level event.
- Maintenance may be completed from the top of the basin as long as the County’s standard backhoe can reach all areas of the basin from the maintenance road. The equipment limitations

require a maximum basin width less than 35 feet wide and a basin depth no greater than 6 feet from the periphery road to the basin invert.

- Except where a joint-use partner is responsible for security, the basin site shall be fenced with chain-link fence in accordance with OCPW Standard Plan 600-3-0C (Orange County Public Works, 2018).

#### 7.2.6.3 Easements and Maintenance Mechanisms

All detention basins require lifetime maintenance. The project owner and design engineer shall consult with OCPW for determination of which maintenance mechanism is required for a particular project. At a minimum, privately owned and maintained detention facilities shall have a recorded easement agreement with a covenant binding on successors, or another mechanism acceptable to OCPW.

R/W for basins should be obtained/provided in fee (or fee simple) unless an easement is approved in writing by OCPW before environmental permitting.

Detention facilities are often located within or adjacent to sensitive environmental areas. The design engineer must investigate which permits might be necessary from various Agencies, including but not limited to:

- U.S. Army Corps of Engineers (Section 404 Wetland Permit)
- U.S. Department of Fish and Wildlife (Section 7 Consultation, Section 10 permits)
- California Department of Fish and Wildlife (Section 1600 Permit)
- Regional Water Quality Control Board (Section 401 Water Quality Certification)
- California Coastal Commission
- California State Water Resources Control Board

These permits are discussed in Chapter 1.6. It is important that the final permits and/or permit conditions allow for the future and perpetual maintenance of a detention facility without the need of returning to the permitting Agency.

### 7.2.7 Basin Ownership

The goal of WQ basins is to improve water quality of surface water and groundwater. Local jurisdictions may reject or require that a proposed WQ measure, including any related hydromodification, be modified in order to ensure that control measures can be reasonably maintained. Dispersed drainage facilities are small scale facilities that typically treat and modify runoff from less than five acres. Typically, these basins are solely dedicated to the purpose of meeting development requirements and are not intended to treat and modify runoff of any watershed area beyond the limits of the development. Therefore, basins that are strictly for WQ purposes shall be deemed to be for private ownership and private maintenance, unless they are accepted for public ownership by County or city.

A maintenance cost agreement may be set up with OCFCD/County of Orange where conjunctive use basins are constructed with a new development. The mere comingling of private water quality flows with public water quality flow does not constitute a shift in the burden of operation and maintenance to OCFCD/County of Orange. In the case of failure of a local owner to comply with applicable regulations, OCFCD/County of Orange reserve the right to perform all necessary corrections, at the private owner’s cost, to bring a basin back into compliance.

### 7.2.8 Conjunctive Use of Detention Facilities

Conjunctive use means the use of a facility for two or more purposes. Because dry detention facilities do not store water between storm events, it is often possible to propose conjunctive uses for detention facilities involving water quality treatment and active or passive recreation. Forebays are recommended to capture and confine sediment and debris in one area of the detention facility to enhance the possibilities for conjunctive use by reducing the scale of maintenance.

Conjunctive use of detention facilities for water quality treatment may be acceptable. When a detention facility is used for both water quality and flood control, the water quality retention volume (e.g., volume for infiltration and wet pond volume) shall be provided in addition to the volume designated for flood storage. Design criteria for water quality facilities is beyond the scope of this manual; the design engineer shall reference the OC DAMP and consult with OC Watershed on appropriate governing agency's stormwater quality manuals and guidance for water quality aspects of detention basin design.

In all cases, the reviewer/designer shall obtain concurrence from OCFCD/County of Orange for all joint uses. Joint uses shall be subordinate to and shall not interfere with OCFCD/County of Orange operation and maintenance activities of the basin as a local drainage facility. OCFCD/County of Orange shall have full discretion in deciding whether facilities may be used by others for conservation, recreation, utility easements, lease, and/or other purposes. Use of land for a detention basin and its conjunctive uses shall be consistent with an adopted specific, general plan, or other legal entitlement. Potential joint uses of the basin property shall be identified, and design shall be compatible with reasonable joint uses. Joint uses shall be noted on the title sheet.

### 7.2.9 Maintenance Cost for Conjunctive Uses

Apportionment of maintenance costs for conjunctive use facilities will require a cooperative agreement which states the obligations of the parties.

### 7.2.10 Water Quality & Hydromodification Considerations

Low Impact Developments (LID) are an integral part of modern-day drainage, their primary goal is exemplified by WQ and hydromodification basins. They store, infiltrate, evaporate, and detain runoff. However, the LDM is primarily concerned (and throughout its chapters) with the provision of 100 yr. flood protection for habitable structures. The subject matter of LID, including WQ and hydromodification, basins falls under the jurisdiction of OC Watershed as it implements the ordinances, rules, and regulations of the California State Water Resources Control Board. County of Orange is governed by different requirements according to the San Diego Regional Water Quality Control Board (in south Orange County) and the Santa Ana Regional Water Quality Control Board (in north Orange County). While, the LDM is not intended to cover, in detail, subject matter pertaining to LID; the LDM categorizes LID basins for the sake of establishing expected maintenance needs.

LID basins route post-development runoff to mimic pre-development hydrology. Their design may include geomorphological factors for the preservation of streambeds. Irrespective of their current methods, LID basins can be categorized into three main and distinct classes:

- Underground detention/retention
- Onsite bioretention, often as BMPs
- Offsite (local, sub-regional, and/or regional) detention or bioretention

The MS4 Permits are updated periodically. Currently, the Permit requires mitigation for potential hydromodification impacts that require flow duration matching of the 2-year storm event for projects that drain to areas susceptible to hydromodification as shown on the mapping provided at OC Environmental Resources website (www.ocwatersheds.com/). OC Watershed's approval is required for matters pertaining to water quality. The Permit also requires low impact development measures such as harvest and reuse, infiltration and biofiltration of the 85$^{th}$ percentile storm event. Since all of these mitigation measures require runoff storage, there is the potential for combining of these facilities with traditional flood mitigation basins. Some of the infiltration, biofiltration, and hydromodification components of the basin may need to be detained longer than the 24-hour standard used for flood mitigation. If a basin does not drain within 24-hours, then the basin shall be analyzed using a multi-day storm. In cases where water quality basins and flood control basins are combined, the engineer shall consult with OC Watersheds for their latest design criteria. Water quality features can be designed as dead storage, not interfering with the active storage or routing, if approved by OC Watersheds. For complete information about water quality design considerations, please refer to OC Environmental Resources Model Water Quality Management Plan (WQMP) and Technical Guidance Document (TGD) for both Santa Ana and San Diego Regional Water Quality Control Boards at: http://www.ocwatersheds.com/documents/wqmp.

## 7.3 Design Procedure – Detention Routing Analysis

The design procedure for detention basin routing includes 1) development of a hydrograph, and 2) routing of the hydrograph through the basin. The appropriate methods for both the hydrograph development and the basin routing are detailed in the Orange County Hydrology Manual and its subsequent addendums.

Because this manual focuses on tributary areas less than 640 acres, the designer must determine whether the small area hydrograph method or the unit hydrograph method (both presented in the Orange County Hydrology Manual) provides the best modeling of the overall system. Generally, if the tributary area is less than 640 acres, and the time of concentration from the rational method is less than 25 minutes, the small area hydrograph procedure (Section J of the Orange County Hydrology Manual, "Small Area Runoff Hydrograph Development") should be used. However, if the tributary area to the basin is less than 640 acres and has a time of concentration of over 25 minutes or the basin is part of a larger (greater than 640 acres) watershed, the unit hydrograph methods from Section E, "The Unit Hydrograph Method," of the Orange County Hydrology Manual may be used.

Regardless of the hydrograph methodology, all basin routing should follow the Orange County Hydrology Manual. The routing for flow-through basins is outlined in Section F of the Hydrology Manual. The routing for flow-by basins is outlined in Section G, "Flow-by Analysis," of the Orange County Hydrology Manual.

## 7.4 Design Procedure – Inlets, Outlet Structures and Spillways

The design procedures for inlets, outlets and spillways require hydraulic design. The following sections describe the hydraulic design requirements as well as debris and erosion design considerations.

### 7.4.1 Inlet Design Procedure

Basin inlets through basin embankments (engineered fill) and cut slopes shall be constructed with reinforced concrete pipe (RCP) or boxes (RCB) and shall conform to the structural requirements in Chapters 4 and 10. Other basin inlets such as concrete weirs, side weirs, gates, sluicegates, chutes, and open channels are also acceptable when designed with the criteria specified herein.

Either a weir or gate(s) may be used for inlet control. The inlet structure design shall be based upon routing the design flood hydrographs through the inlet(s) and basin or shall be sized to deliver peak discharge with the basin full.

Stilling basins or energy dissipator structures shall be constructed to control basin invert erosion. Stilling basins and energy dissipators shall be designed assuming peak inlet flow with the basin empty. Energy dissipator structures shall be designed in accordance with the following:

- U.S. Bureau of Reclamation. *Design of Small Dams (Chapter 9, Section E, Latest Edition)*.
- U.S. Bureau of Reclamation. *Hydraulic Design of Stilling Basins and Energy Dissipators (Latest Edition*).
- U.S. Army Corps of Engineers. EM 1110-2-1602, *Hydraulic Design of Reservoir Outlet Works (Chapter 5, Latest Edition)*.
- An alternative approach approved by OCPW

See Chapter 8, “Energy Dissipation,” for more information.

Inlets with flow velocities less than 5.0 fps may use OCPW Standard Plan 1326 in lieu of the design references listed above. Under no circumstances shall an incoming stream, channel, or other inlet discharge flow with velocities greater than 5.0 fps onto an unprotected/unlined basin slope or invert. In addition, the reviewer/designer should ensure that all inlet flows are able to be safely and effectively routed to the basin outlet without causing unacceptable erosion or other damage to the basin or requiring above normal basin maintenance as determined by OCPW.

If a side weir is proposed, the side weir elevation and length shall be optimized considering: the need to limit premature spill to the basin, the need to limit the maximum bypass discharge to match the downstream channel’s safe conveyance capacity, and cost. Either the side weir shall be designed for stable subcritical flow in a concrete rectangular feeder (bypass) channel, or the design must be verified by an approved physical model. Side weir design and flood routing shall be analyzed using the spatially varied flow equation when channel velocities exceed 5 feet per second.

Equations or methods used to calculate the side weir coefficient of discharge, Cd, such as those by Hager (1987), may be considered for approval by OCPW provided that adequate documentation is included to substantiate the proposed analyses and equations. However, under no circumstances will side weir analyses using equations other than the spatially varied flow equation (Chow, 1959) be considered as adequate for channel velocities above 5 feet per second.

Inlet gates within a bypass channel may be applicable where basin volume is limited and must be conserved by preventing premature or excessive spill into the basin, where close downstream control can be maintained (i.e., the gates are controlled by downstream water surface), and where sediment and debris are not problems. Due to the labor associated with operating and maintaining gates, OCPW will only approve inlet – gate operated basins on a case-by-case basis when no other inlet alternative appears feasible. If gates are proposed, standby power and telemetry of gate, basin and channel status shall be provided, and control shall have one level of redundancy. The basin volume shall include consideration of the uncertainty in estimating weir and gate flow rates by verification through a mathematical, and, if necessary, a physical hydraulic model study.

If a trash rack is proposed at a basin inlet, the reviewer/designer should assume a minimum 50% blockage of the grate openings for determining the hydraulic adequacy of the upstream conveyance system. Trash racks shall be located at the upstream end of conduits. Sediment and debris volume and maintenance schedules shall be determined as set forth in Chapter 9 and Chapter 7.2.6 of this manual.

Where sediment basins and debris structures cannot be constructed or are impractical, sediment and debris volume shall be added to the basin as set forth in Chapter 9 of these guidelines. Basin inlets shall be designed such that large volumes of debris will not enter the basin. Since large volumes of debris may cause plugging of the basin inlet and outlet structures, the design of the basin inlets shall consider the installation of debris and/or sediment catching devices upstream of the basin inlet structure or at designated areas on the basin invert. For the design of debris posts, debris deflectors, or trash racks, refer to the Los Angeles County Flood Control District, Design Manual Debris Dams and Basins or Chapter 9 of this manual. For basins where debris inflow is unavoidable or where un-maintained or rarely maintained vegetative growth may be expected, basin outlet structures shall include trash racks or other debris catching devices to prevent plugging. Alternatively, the basin outlet shall be designed of sufficient size, such that plugging by debris will not be an issue. See Section 7.4.2, below.

### 7.4.2 <u>Outlet and Spillway Design Considerations</u>

A low-level outlet is required for emptying or lowering the reservoir in case of an emergency; for inspection and maintenance of the dam, basin invert, and appurtenances; and for releasing waters to meet downstream water rights, where applicable.

Only three types of outlet conduits are allowed -- precast reinforced concrete pipe, box structures and cast-in-place reinforced concrete box structures.

RCP in facilities not subject to DSOD jurisdiction shall be constructed in bedding conforming to the requirements of OCPW Standard Plan 1319 unless recommended otherwise by a geotechnical engineer or the OCPW Materials Laboratory.

Outlet conduits that pass beneath or through basin slopes or embankments shall include design components to eliminate the possibility of piping or migrating soil particles along the length of the conduit. Expansion joints in the outlet conduit, spaced at a maximum of 32 feet, are required for compressible foundations. Unless otherwise authorized by OCPW, all pipe joints not encased in concrete shall be water tight through the dam embankment.

All outlet conduits shall be designed for internal pressure equal to the full reservoir head and for superimposed embankment loads, acting separately. Embankment loads should be computed in accordance with Marston’s Theory or another similarly accepted method. Internal liners of conduits shall not be considered as adding structural strength. For basins subject to sediment and debris inflow, trash racks or other debris catching devices shall be installed upstream (debris posts and deflectors) or at the entrance (trash racks) to the basin outlet structure as required, or the basin outlet structure shall be of a sufficient size that clogging from debris is not an issue. For the design of debris posts, debris deflectors, or trash racks, refer to the Los Angeles County Flood Control District, Design Manual Debris Dams and Basins or this manual, Chapter 9. Trash rack bars and supports shall be structurally designed for a minimum of 100 % of the basin storage head to which they will be subjected if completely clogged. The basin outlet structure shall be designed to deliver the design outlet discharges when the design debris and sediment reaches 100% of the estimated sediment volume.

The hydraulic capacity (cross sectional area of the openings) of all trash racks (the combined grate openings) shall be a minimum of 200% of the hydraulic capacity of the outlet structure conduit to allow for the possibility of clogging and plugging.

For basins subject to heavy debris inflow, debris posts or deflectors should be designed upstream of inlet structures or if unfeasible, around the outlet structure at a distance not to hinder the operation of basin outflows. The size, setback distance from the inlet, and spacing of the debris posts or deflectors should be designed on a case-by-case basis. If appropriate, the basin shall include extra storage for a debris pool of sufficient depth to protect the basin outlet structure.

Riser outlet structures shall be designed to permit the design discharge at sediment pool's highest predicted elevation with provisions for discharging water at lower elevations to satisfy the functional requirements of the structure. Riser outlet structures shall be designed neglecting the riser slots for basin storage volume and discharge but should be taken into account when determining downstream facility capacity and sizing. The riser spillway should be designed per Section 9.26, "Drop Inlet (Shaft or Morning Glory) Spillways," Design of Small Dams, U.S. Bureau of Reclamation or EM1110-2-1603, Section III, Paragraph 5-9 and accompanying HDC 140-1 to 140-1/8 or an alternative method approved by OCPW.

The channel downstream of the basin outlet must be lined to prevent scour until the basin discharge energy is reduced to an acceptable non-erosive velocity.

### 7.4.3 Outlet and Spillway Design Procedures

Spillways and outlets are generally designed using either culvert, orifice or weir flow hydraulic principals. Chapter 5 discusses culvert hydraulics. The following sections include information on orifice calculations and weir calculations for sharp-crested and broad crested weirs. The Urban Drainage Design Manual (HEC 22) published by the Federal Highway Administration provides information on orifice and weir flow calculations.

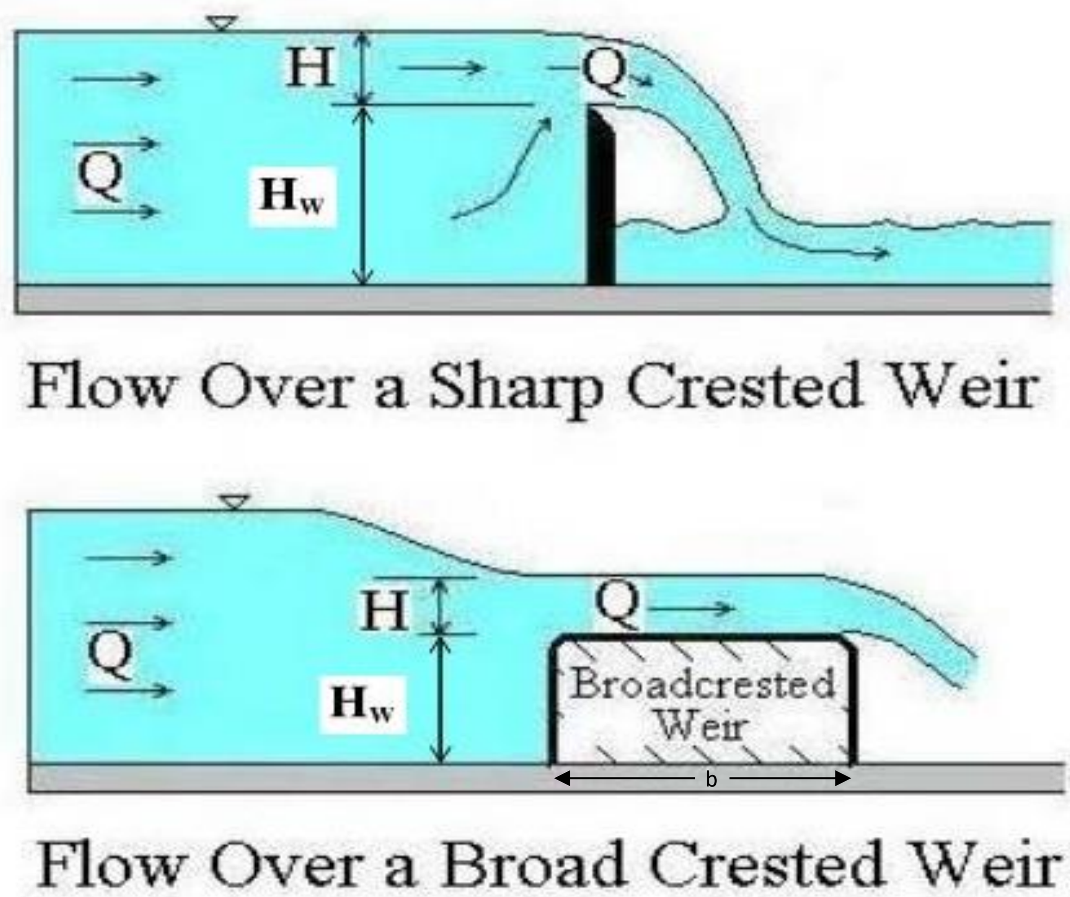



**Figure 7-2: Sharp and Broad Crested Diagrams (Bengtson, 2009)**

### 7.4.3.1 Sharp-Crested Weirs

Sharp-crested weirs have a relatively thin crest such that water will tend to develop a nappe as it flows over the crest (Figure 7-3). The capacity of a sharp-crested weir depends on the influence of end contractions. For a sharp-crested weir with no contractions, flow is calculated with the following equation:

$$Q = C_{scw} L H^{3/2}$$

Where:

| | | |
|---|---|---|
| $Q$ | = | flow over weir crest (cfs) |
| $C_{scw}$ | = | sharp-crested weir coefficient |
| $L$ | = | length of weir crest (ft) |
| $H$ | = | head above of weir crest, excluding velocity head (ft) |

The weir coefficient $C_{SCW}$ varies with the ratio of hydraulic head above the weir and the height of the weir ($H/H_W$). For U.S. traditional units, the weir coefficient can be calculated as:

$$C_{scw} = 3.27 + 0.4\frac{H}{H_W}$$

Where:

| | | |
|---|---|---|
| $C_{scw}$ | = | sharp-crested weir coefficient |
| $H$ | = | head above of weir crest, excluding velocity head (ft) |
| $H_W$ | = | height of the weir crest (ft) |

A sharp-crested weir with two end contractions can be analyzed using the equation:

$$Q = C_{scw}\,(L - 0.2H)H^{3/2}$$

Where:

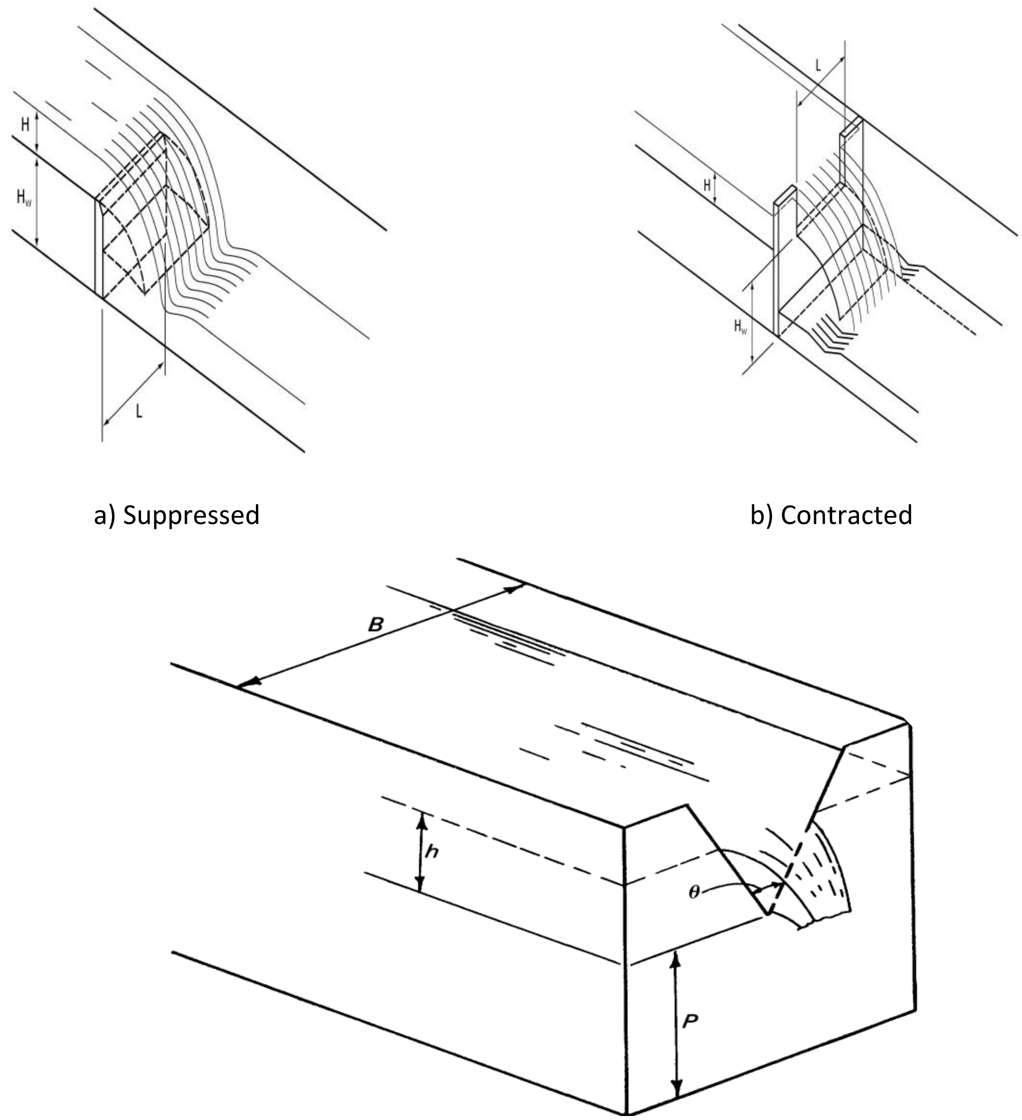
| | | |
|---|---|---|
| $Q$ | = | flow over weir crest (cfs) |
| $C_{scw}$ | = | sharp-crested weir coefficient |
| $L$ | = | length of weir crest (ft) |
| $H$ | = | head above of weir crest, excluding velocity head (ft) |

When the tailwater behind a sharp-crested weir rises above the weir crest elevation, the submerged condition will reduce the discharge over the weir. The equation for a submerged sharp-crested weir is:

$$\frac{Q_s}{Q} = \left[1 - \left(\frac{H_2}{H_1}\right)^{3/2}\right]^{0.385}$$

Where:

| | | |
|---|---|---|
| $Q_s$ | = | flow over submerged sharp-crested weir (cfs) |
| $Q$ | = | flow over sharp-crested weir under un-submerged conditions with same upstream headwater (cfs) |
| $H_1$ | = | head above of weir crest upstream of crest, excluding velocity head (ft) |
| $H_2$ | = | head above of weir crest downstream of crest, excluding velocity head (ft) |




c) V-Notch

**Figure 7-3: Sharp Crested Weir Configurations (County of San Diego, 2014, Figure 6-4)**

### 7.4.3.2 V-Notch Weirs

V-notch weirs are a particular type of sharp-crested weir with a triangular cross-section. The discharge through a v-notch weir can be calculated from the following equation:

$$Q = 2.5 * \tan\left(\frac{\theta}{2}\right) H^{5/2}$$

Where:

| | | |
|---|---|---|
| $Q$ | = | low over weir crest (cfs) |
| $\theta$ | = | angle of v-notch (degrees) |
| $H$ | = | depth of water above apex of v-notch (ft) |

### 7.4.3.3 Submerged Weirs

Submerged weirs occur when the downstream tail water is above the weir crest height ($H_C$). Figure 7-4 illustrates a submerged weir condition. A submerged weir results in reduced discharge over the weir.

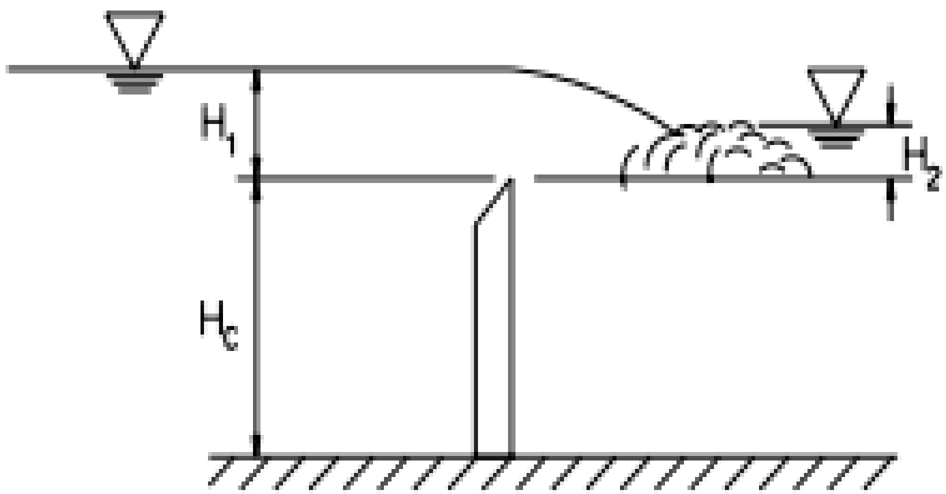



**Figure 7-4: Submerged Weir Configuration (Connecticut Dept of Transportation, 2000)**

The submerged discharge equation for a sharp crested weir is:

$$Q_s = Q_r \left(1 - \left({H_2}/{H_1}\right)^{1.5}\right)^{0.385}$$

Where:

| | | |
|---|---|---|
| $Q_s$ | = | Submerged flow over weir crest (cfs) |
| $Q_r$ | = | Unsubmerged flow over weir crest (cfs) |
| $H_1$ | = | Upstream head above crest (ft) |
| $H_2$ | = | Downstream head above crest (ft) |

### 7.4.3.4 Broad-Crested Weir

In most cases, spillways traversing the top of an embankment are best modeled as free-flowing broad-crested weirs. The equation for evaluating flow over a broad-crested weir is:

$$Q = C_{BCW} L H^{3/2}$$

Where:

| | | |
|---|---|---|
| $Q$ | = | flow over weir crest (cfs) |
| $C_{BCW}$ | = | broad-crested weir coefficient |
| $L$ | = | length of weir crest (ft) |
| $H$ | = | head above of weir crest, excluding velocity head (ft) |

The determination of the broad crested weir coefficient requires that the water surface elevation is measured upstream of the weir crest a distance of at least 2.5 times the head (2.5H) above the weir crest elevation. For submerged weirs where downstream water levels are high enough to affect the discharge, the procedures for calculating discharge coefficients shall be per the US Bureau of Reclamation *Design of Small Dams* 3rd Edition, 1987. Table 7-1 provides broad-crested weir coefficients based on the effective head over the weir and the breadth of the weir. A typical roadway crossing can be modeled as a broad-crested weir with a weir coefficient $C_{BCW}$=2.6; for other applications, a broad-crested weir coefficient of $C_{BCW}$=3.0 is usually appropriate.

| **Broad Crested Weir Coefficients, $C_{BCW}$** | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| *Measured Head* (H) (ft)* | *Weir Breadth, b (ft)* | | | | | | | | | | |
| | *0.50* | *0.75* | *1.0* | *1.5* | *2.0* | *2.5* | *3.0* | *4.0* | *5.0* | *10.0* | *15.0* |
| *0.2* | 2.80 | 2.75 | 2.69 | 2.62 | 2.54 | 2.48 | 2.44 | 2.38 | 2.34 | 2.49 | 2.68 |
| *0.4* | 2.92 | 2.80 | 2.72 | 2.64 | 2.61 | 2.60 | 2.58 | 2.54 | 2.50 | 2.56 | 2.70 |
| *0.6* | 3.08 | 2.89 | 2.75 | 2.64 | 2.61 | 2.60 | 2.68 | 2.69 | 2.70 | 2.70 | 2.70 |
| *0.8* | 3.30 | 3.04 | 2.85 | 2.68 | 2.60 | 2.60 | 2.67 | 2.68 | 2.68 | 2.69 | 2.64 |
| *1.0* | 3.32 | 3.14 | 2.98 | 2.75 | 2.66 | 2.64 | 2.65 | 2.67 | 2.68 | 2.68 | 2.63 |
| *1.2* | 3.32 | 3.20 | 3.08 | 2.86 | 2.70 | 2.65 | 2.64 | 2.67 | 2.66 | 2.69 | 2.64 |
| *1.4* | 3.32 | 3.26 | 3.20 | 2.92 | 2.77 | 2.68 | 2.64 | 2.65 | 2.65 | 2.67 | 2.64 |
| *1.6* | 3.32 | 3.29 | 3.28 | 3.07 | 2.89 | 2.75 | 2.68 | 2.66 | 2.65 | 2.64 | 2.63 |
| *1.8* | 3.32 | 3.32 | 3.31 | 3.07 | 2.88 | 2.74 | 2.68 | 2.66 | 2.65 | 2.64 | 2.63 |
| *2.0* | 3.32 | 3.31 | 3.30 | 3.03 | 2.85 | 2.76 | 2.27 | 2.68 | 2.65 | 2.64 | 2.63 |
| *2.5* | 3.32 | 3.32 | 3.31 | 3.28 | 3.07 | 2.89 | 2.81 | 2.72 | 2.67 | 2.64 | 2.63 |
| *3.0* | 3.32 | 3.32 | 3.32 | 3.32 | 3.20 | 3.05 | 2.92 | 2.73 | 2.66 | 2.64 | 2.63 |
| *3.5* | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.19 | 2.97 | 2.76 | 2.68 | 2.64 | 2.63 |
| *4.0* | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.07 | 2.79 | 2.70 | 2.64 | 2.63 |
| *4.5* | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 2.88 | 2.74 | 2.64 | 2.63 |
| *5.0* | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.07 | 2.79 | 2.64 | 2.63 |
| *5.5* | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 2.88 | 2.64 | 2.63 |

* H measured at least 2.5H upstream of weir.

**Table 7-1: Broad Crested Weir Coefficients ($C_{BCW}$) (Brown, S.A., et al., 2009, pp 8-27)**

When using the broad-crested weir model to evaluate the capacity of a spillway with a rectangular or trapezoidal cross-section, the length of the weir crest is set as the base width of the spillway channel (*b*). The velocity of flow over the crest spillway is calculated as it passes through critical depth at the control section:

$$v_c = 3.18 \left(\frac{Q}{b}\right)^{1/3}$$

Where

| | | |
|---|---|---|
| $v_C$ | = | critical velocity (fps) |
| $Q$ | = | discharge over the weir (cfs) |
| $b$ | = | length of weir crest (ft) |

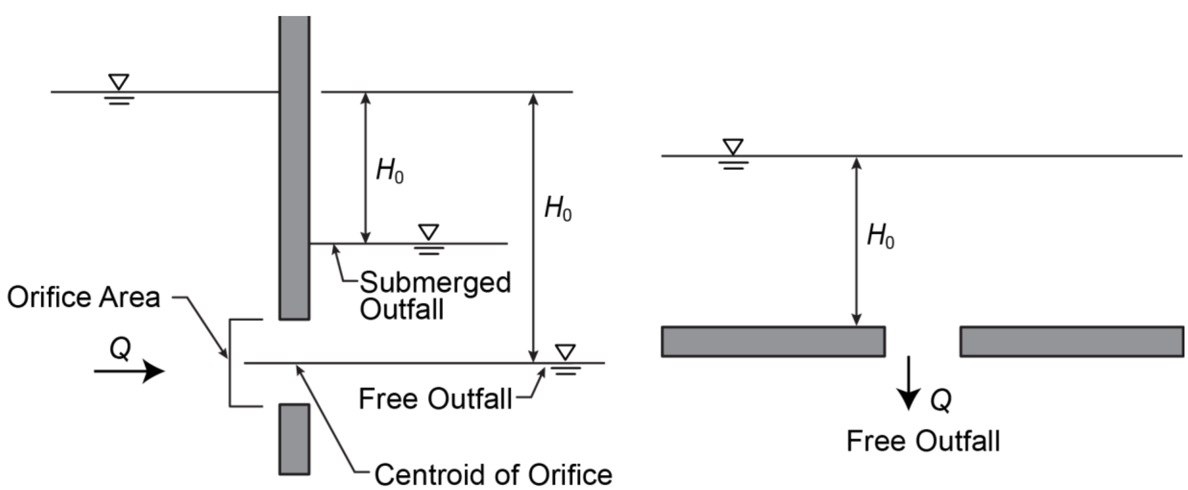
In broad crested weirs where there is potential for cavitation, an Ogee (S-Shaped) spillway should be considered. Ogee spillway design shall be designed per the US Bureau of Reclamation *Design of Small Dams* 3rd Edition, 1987.

### 7.4.3.5 Orifices

A vertical orifice is a circular or rectangular opening, often located in a headwall or the sidewall of a riser structure. Figure 7-5 illustrates typical orifice configurations. The flow rate depends on the submergence of the orifice, effective height of the water above the center of the opening, and the size, shape, and edge treatment of the orifice. For a single submerged orifice, the discharge can be determined using the standard orifice equation:

$$Q = C_o A_o \sqrt{2g(H_o)}$$

Where
$Q$ = orifice flow discharge (cfs)
$C_O$ = orifice discharge coefficient
$A_O$ = cross-sectional area of flow through the orifice (ft$^2$)
$g$ = gravitational acceleration (32.2 ft/s$^2$)
$H_O$ = effective head above orifice (ft)




a) Vertical Orifice b) Horizontal Orifice

**Figure 7-5: Orifice Configurations (County of San Diego, 2014, Figure 6-5)**

When the orifice is unsubmerged, the effective head $H_O$ is measured from the centerline of the orifice to the upstream water surface elevation. For submerged orifices, the effective head is the difference in elevation of the upstream and downstream water surfaces.

When the orifice has sharp, clean edges (i.e., the material is thinner than the orifice diameter), an orifice discharge coefficient ($C_O$) of 0.6 is appropriate. For sharp, ragged edged orifices, such as those produced by cutting openings in corrugated pipe with an acetylene torch, a value of $C_O$=0.40 should be used. The orifice coefficient should also be adjusted when the diameter of the orifice approaches the thickness of the orifice plate. Table 7-2 summarizes orifice discharge coefficients for different edge conditions. Pipes smaller than 1 foot in diameter may be analyzed as submerged orifices, as long as there is adequate headwater ($H_O/D$>1.5). Pipes larger than 1 foot in diameter are more appropriately analyzed as culverts (see Chapter 5). Flow through multiple orifices may be computed by summing the flow through the individual orifices.

| **Edge Condition** | **Orifice Coefficient, $C_O$** |
|---|---|
| Sharp, Clean Edge (t < d) | 0.60 |
| Sharp, Ragged Edge (t < d) | 0.40 |
| Thick, Squared Edge (t > d) | 0.80 |
| Thick Rounded Edge (t > d) | 0.92 |

Note: t is thickness, d is diameter of orifice

**Table 7-2: Orifice Coefficients**

### 7.4.3.6 Riser Structures

A riser structure is a general term for structures having inlet openings that are parallel to the water surface in the detention facility. Riser structures with circular cross-section are often called standpipes, and rectangular riser structures are often called inlet boxes. The hydraulic behavior of flow through a riser structure changes and must be analyzed differently depending on the stage in the basin. Flow through a riser structure generally proceeds through four phases:

1) riser weir flow control
2) riser orifice flow control
3) barrel inlet flow control
4) barrel pipe flow control

When the water surface reaches the top edge of the riser, flow will typically begin to pass through the structure in the manner of a sharp-crested weir (for sharp-crested weir equation and coefficients, see Section 7.4.3.1), with a crest length equivalent to the perimeter of the riser structure.

As the depth of water increases and submerges the top of the riser, the flow will transition to an orifice-type flow. This horizontal orifice flow depends on the area of the top of the riser structure. Refer to Section 7.4.3.5 Orifices for the orifice flow equation.

The transition zone between weir and orifice flow for riser structures is not well defined. Though the transition from weir flow to orifice flow is gradual, it is commonly assumed to occur at a discrete water surface elevation to simplify the analysis. The transition water surface elevation ($H_o$=$h_T$) is found by calculating the point at which the weir equation and orifice equation yield the same discharge:

$$H_o = \sqrt{2g}\,\frac{C_O A_O}{C_{SCW} L}$$

Where:

| | | |
|---|---|---|
| $H_o$ | = | effective head above riser pipe entrance at transition to orifice flow(ft) |
| $g$ | = | acceleration due to gravity (ft/s$^2$) |
| $C_0$ | = | orifice coefficient |
| $A_0$ | = | cross-sectional area of the riser pipe entrance (ft$^2$) |
| $C_{SCW}$ | = | sharp crested weir coefficient |
| $L$ | = | riser pipe circumference as weir crest length (ft) |

Thus, the weir equation is used for calculating flow through a riser structure for water surface elevations $h \le h_T$ and the orifice equation for water surface elevations $h > h_T$. As the water surface elevation rises further, the control can change to barrel inlet flow control and/or barrel pipe flow control. General best practice is to ensure that the outlet barrel has greater capacity than the riser structure under design conditions. The USBR *Design of Small Dams* (1987) discusses related topics in more detail and may be used in the design of riser structures.

### 7.4.3.7 Perforated Risers

Perforated risers are a special case of orifice flow that can be used to obtain extended detention times. As such, they are often useful in water-quality treatment applications. Holes are normally spaced a minimum of three to four orifice diameters (center to center) apart, limiting the number of holes such that they do not compromise the overall integrity of the riser.

Assuming the riser is constructed of a relatively thin material, the perforations will operate as orifices. Therefore, the discharge through the orifices on the perforated risers is equivalent to the summation of the flow through individual orifices in the riser. The design engineer shall use care when specifying perforated risers, since they are often subject to clogging, and measures to reduce such clogging such as gravel jackets and/or wire mesh also have implications for the maintenance of the perforated riser.

### 7.4.3.8 Combination Outlets

Combinations of culverts, weirs, orifices, and riser structures can provide multiple-stage outlet control for different control volumes and storm frequencies. These combination outlets may have independent outlet controls, but often outlet structures will share a common outlet. Combination outlets require composite stage-discharge curves based on the hydraulic performance curves of the component outlet structures. The total discharge from the outlets will generally be the summation of its individual outlets, constrained by the capacity of common outlet conduits and possibly tailwater conditions.

### 7.4.3.9 Buoyancy

Buoyancy creates uplift forces that can damage detention basin riser structures. Outlet structures shall be anchored properly such that they will withstand buoyant forces. The design engineer shall consider resistance to buoyant forces both for the anchoring of the system as a whole and for major connecting components (e.g., band couplings) of the outlet structure. The design condition shall assume maximum design water surface elevation in the basin with no water inside the outlet structure.

## 7.5 Acceptable Software

The following are acceptable software applications for detention basin design hydrology and hydraulic calculations:

1. HEC-RAS by USACE
2. Advanced Engineering Software (AES) for Hydrograph Development and Basin Routing
3. OCFCD's SIDE WEIR program for Side Weir Analysis or derivative programs

If a computer application other than those listed above is used, documentation needs to be provided to OCPW with justification for its use. Additional review and permitting fees may be required for the County to hire specialized expertise to perform an adequate review.

## 7.6 References

Bengtson, Harlan. (2009). *Measuring Open Channel Flow Rates with a weir or a flume*. Bright Hub Engineering. www.brighthubengineering.com/hydraulics-civil-engineering/51435-introduction-to-the-weir-and-flume/

California Department of Water Resources. (n.d.). *Jurisdictional Sized Dams*. Retrieved June 29, 2020, from https://water.ca.gov/Programs/All-Programs/Division-of-Safety-of-Dams/Jurisdictional-Sized-Dams

Chow, V. T. (1959). *Open-Channel Hydraulics*. McGraw-Hill.

Connecticut Department of Transportation. (2000). *Drainage Manual*.

County of Orange, Cities of Orange County, & Orange County Flood Control District. (2003). *Drainage Area Management Plan (DAMP)*. https://www.ocwatersheds.com/documents/damp/mapplan

County of San Diego Department of Public Works Flood Control Section. (2005). *Drainage Design Manual*.

County of San Diego, Department of Public Works Flood Control Section. (2014). San Diego County Hydraulic Design Manual.

Hagar, W. H. (1987). Lateral Outflow Over Side Weirs. *ASCE Journal of Hydraulic Engineering*, *113*(4). https://doi.org/10.1061/(ASCE)0733-9429(1987)113:4(491)

Los Angeles County Flood Control District. (n.d.). *Design Manual Debris Dams and Basins*. http://dpw.lacounty.gov/DES/design_manuals/

Orange County Environmental Management Agency. (1986). *Orange County Hydrology Manual*.

Orange County Public Works. (1986). *Orange County Hydrology Manual (Orange County Hydrology Manual Addendum No. 1)*.

Orange County Public Works. (2018, September). *Standard Plans (2018 edition)*.

Peterka, A. J. (1984). *Hydraulic Design of Stilling Basins and Energy Dissipators*. Engineering Monograph No. 5. U.S. Bureau of Reclamation.

U.S. Army Corps of Engineers. (1980). Engineering Manual No. 1110-2-1602, Engineering and Design, *Hydraulic Design of Reservoir Outlet Works (Chapter 5)*.

U.S. Bureau of Reclamation. (1987). *Design of Small Dams (3rd Edition)*.

# Chapter 8 Energy Dissipation

## 8.1 Introduction

The construction of storm drains and various types of drainage conveyance associated with property development traditionally resulted in modifications to drainage characteristics, including increases in peak discharges and velocities of runoff. Storm drain discharges into unlined channels or natural watercourses have the potential to cause erosion. This chapter focuses on riprap aprons, which are the most common type of energy dissipator, but also summarizes several other methods for energy dissipation. The designer is encouraged to investigate referenced sources for further discussion of energy dissipators.

## 8.2 Design Criteria

Energy dissipation is required when a project increases the exit velocity and turbulence at an outlet above the existing (pre-project) condition. However, if the velocity does not have the potential to be erosive, energy dissipation is not required. Energy dissipation may also be required when a project proposes to concentrate surface runoff into discrete discharge points (i.e., concentrating sheet flow and discharging into a stream via a down-drain, etc.). Energy dissipation shall reduce the velocity to non-erosive levels. The dissipation design event shall be the same as for the immediately upstream facility.

## 8.3 Hydraulic Design

### 8.3.1 Riprap Aprons

Where a storm drain discharges into a natural channel and pipe flow velocity is less than 20 fps, outlet scour protection alone may be considered, but it must be shown that flow velocities are reduced to non-erodible levels.  This may require a geotechnical engineer’s consultation.

The outlet scour protection may consist of concrete, riprap, or grouted rock, but shall not adversely impact the channel flow. The minimum length of the scour protection downstream of conduit outlet shall be the procedure outlined in the FHWA *Hydraulic Design of Energy Dissipators for Culverts and Channels* (HEC-14).

To find the appropriate riprap apron length and depth, first the riprap size ($D_{50}$) must be found.

Using equation 10.4 from HEC-14 (Thompson and Kilgore, 2006):

$$D_{50} = 0.2D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{4/3}\left(\frac{D}{TW}\right)$$

Where:

| | | |
|---|---|---|
| $D_{50}$ | = | Particle side or gradation, of which 50 percent, of the mixture is finer by weight (ft) |
| $Q$ | = | Design discharge (cfs) |
| $D$ | = | Culvert diameter (circular) (ft) |
| $g$ | = | gravity, 32.2 (ft/s$^2$) |
| $TW$ | = | Tailwater Depth (ft) |

If the tailwater is unknown assume it is equal to 0.4D, the equation for $D_{50}$ is as follows:

$$D_{50} = \frac{0.049Q^{\frac{4}{3}}}{D^{\frac{7}{3}}}$$

Where:

$Q$ = Design discharge (cfs)

$D$ = Culvert diameter (circular) (ft)

The equation is reduced to Table 8-1

| Diameter, D (in) | $D_{50}$(ft) |
|---|---|
| 18 | 0.0190*$Q^{\frac{4}{3}}$ |
| 24 | 0.0097*$Q^{\frac{4}{3}}$ |
| 30 | 0.0058*$Q^{\frac{4}{3}}$ |
| 36 | 0.0038*$Q^{\frac{4}{3}}$ |
| 42 | 0.0026*$Q^{\frac{4}{3}}$ |
| 48 | 0.0019*$Q^{\frac{4}{3}}$ |
| 54 | 0.0015*$Q^{\frac{4}{3}}$ |
| 60 | 0.0011*$Q^{\frac{4}{3}}$ |

**Table 8-1: Riprap $D_{50}$ Calculations**

Once $D_{50}$ is found, the correct measurements for the apron can be found using the following table from the FHWA HEC-14.

| $D_{50}$(in) | Apron Length | Apron Depth |
|---|---|---|
| 10 | 5D | $2.4D_{50}$ |
| 14 | 6D | $2.2D_{50}$ |
| 20 | 7D | $2.0D_{50}$ |
| 22 | 8D | $2.0D_{50}$ |

**Table 8-2: Riprap Apron Length and Depth**

Where conduit discharges to a prismatic channel, OCPW Standard Plan 1326 shall be used. The designer should wrap filter fabric to top of rock along all soil interfaces.

#### 8.3.1.1 Example: <u>Riprap Apron</u>

**Given:**

Q = 85 $ft^3/s$

D = 36 inches

**Find:**

Determine $D_{50}$ and the dimensions for the riprap apron.

**Solution:**

Calculate $D_{50}$:

$$D_{50} = \frac{0.049Q^{\frac{4}{3}}}{D^{\frac{7}{3}}}$$

$$D_{50} = \frac{0.049(85)^{\frac{4}{3}}}{(3)^{\frac{7}{3}}} = 1.41\ ft = 16.93"$$

Using Table 8-2, look up apron length and round $D_{50}$ (16.93 in) to the tabular value equal or greater than (20 in).

**$D_{50}$ = 20 (in)**

Estimate riprap apron dimensions.

Apron length:

L=7D

L = 7(3) = 21 ft

**L=21 (ft)**

Apron Depth:

$D_{apron}$=2.0$D_{50}$

$D_{apron}$= 2.0(20) = 40 in = 3.32 ft

**D=3.32 (ft)**

Width (at apron end):

W=3D + (2/3) L

W= 3(3) + (2/3) (21) = 23 ft

**W=23 (ft)**

### 8.3.2 Stilling Basins

There are additional types of stilling basins, and it is beyond the scope of this manual to provide detailed information on each of them. Information about their proper application and design can be obtained from FHWA HEC-14, the U.S. Army Corps of Engineers' *Hydraulic Design Criteria* and *Engineer Manuals*, the Bureau of Reclamation's *Design of Small Canal Structures*, and other references. The use of riprap, Saint Anthony Falls (SAF) and Colorado State University (CSU) stilling basins are allowed, subject to the approval of OCPW.

### 8.3.3 Dissipator Rings

The use of dissipator rings in pipes, or bars in box culverts, can be an efficient manner to reduce storm drain velocities. Section 7.1 of FHWA HEC-14 (Thompson & Kilgore, 2006) provides a complete discussion of the use and design of dissipator rings at downstream outlets to minimize storm drain exit velocities. Section 7.2 of FHWA HEC-14 provides a complete discussion of the use and design of dissipator rings (velocity control rings) to increase resistance through the length of pipe, thus reducing velocities along the length of the storm drain. A low flow system within the conduit should be provided.

This manual limits the use of dissipator rings in urban applications to velocities greater than 20 fps, where no significant bed loads are anticipated, or where other methods of energy dissipation are impracticable. Exceptions to the aforementioned criteria requires OCPW approval., *Standard Plans for Public Works* (Greenbook) Standard Plan 383-2 (Public Works Standards, Inc., 2009) has an example of a precast pipe dissipator. Dissipator rings shall be modified to facilitate drainage behind the rings and prevent ponding within the culvert.

### 8.3.4 Impact Basin

Where outlet discharge velocities are greater than 20 fps or cannot be reduced to non-erosive with outlet scour protection alone, an energy dissipator shall be specified.

Drop manholes or cleanouts shall not be used for energy dissipators unless, for a special condition, a special structural design is approved. These should be very rare installations.

The Design Engineer shall provide adequate cross sections and topography (generally 200' downstream or 50 times the diameter of pipe, whichever is greatest) to OCPW for review.

Drivable access to the outlet shall be provided for maintenance. Fencing shall be provided as required by Cal OSHA. A protection barrier shall be provided as required by OCPW (see "Protective Barriers" in Chapter 4.5.3 to determine when a protective barrier is necessary).

A U.S. Bureau of Reclamation (USBR) Type VI stilling [impact] basin, as shown in Figure 8-1, is one type of energy dissipator that may be used. The basin dimensions can be determined using Figure 8-2 once the Froude number is known. Table 8-3 and Table 8-4 provide the dimensions for Figure 8-1; it may only be specified for velocities less than 30 fps and discharges less than 400 cfs. This stilling basin is not recommended where debris build-up may cause substantial clogging.

Although tailwater is not necessary for successful operation, a moderate depth of tailwater will improve the performance. For best performance, set the basin so that the maximum tailwater does not exceed: $h_3 + \left({}^{h_2}/_{2}\right)$.

The structural walls shall be designed based upon recommendations of the geotechnical report. Interior surfaces of the structure shall provide for impact flows with extra cover for reinforcing as specified in the Orange County Flood Control District Design Manual.

Riprap should be placed downstream of the dissipator with length per Section 8.3.1 Riprap Aprons. Toe down the downstream end of this rock to minimum depth of 3', with consideration to specific application, including outlet velocity, streambed material, and downstream slope.

The entrance pipe may be tilted downward up to 15°. For greater conduit slopes, use a horizontal or sloping pipe (up to 15°) for at least four conduit widths immediately upstream of the stilling basin.

When a hydraulic jump is expected to form in the downstream end of the pipe and the entrance is submerged, a vent approximately one-sixth the pipe diameter should be installed at a convenient location upstream from the jump.

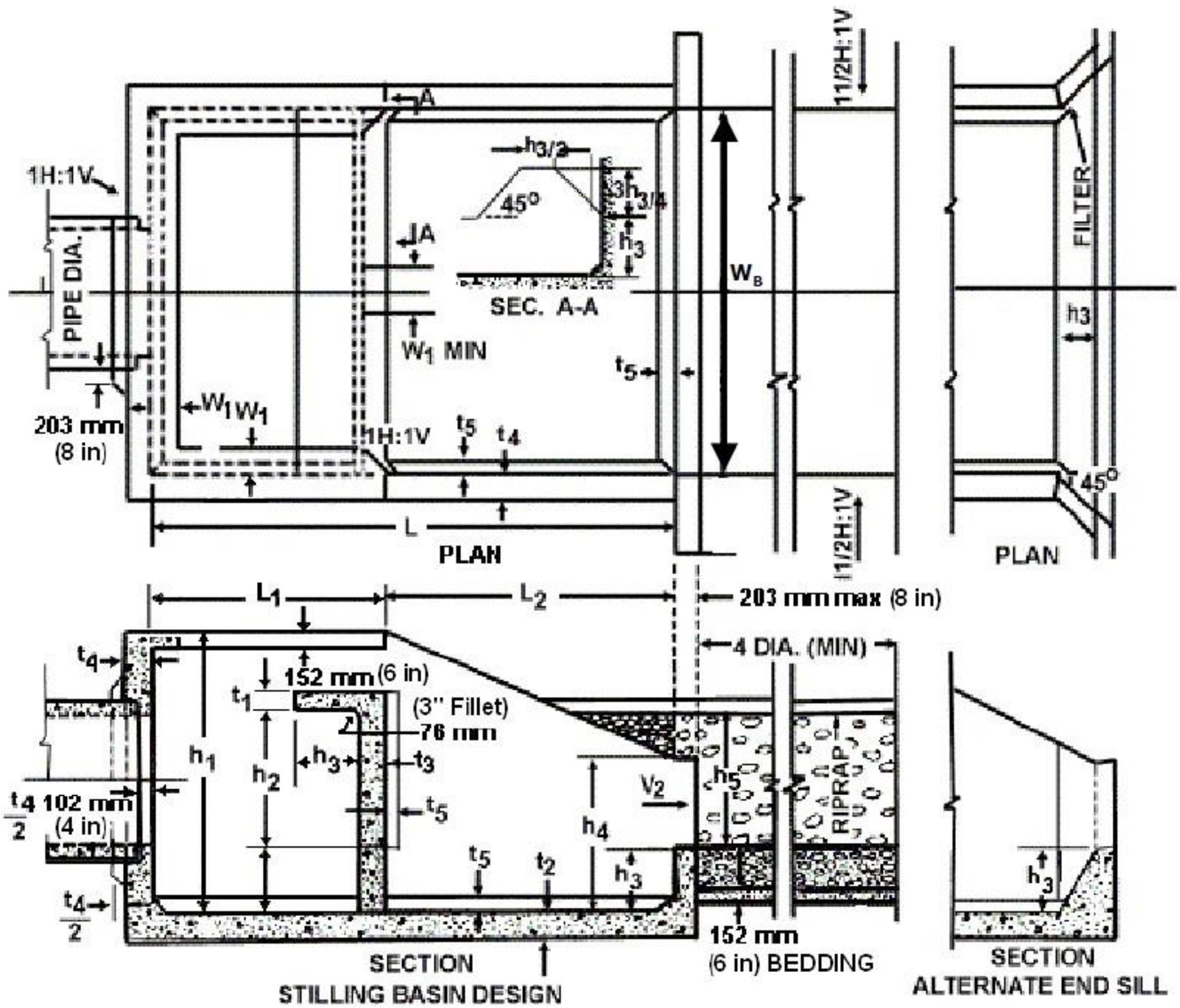



**Figure 8-1: USBR Type VI Basin (Thompson & Kilgore, 2006)**

#### 8.3.4.1 Design Procedure: USBR Type VI Impact Basin

The design of USBR Type VI stilling [impact] basin, as accepted by OCPW, is as follows:

1) Calculate the flow area from the design discharge and velocity entering the dissipator.
2) Calculate the equivalent depth of flow entering the dissipator from a pipe or irregular shaped conduit:

   $y_e = (A/2)^{0.5}$ cross area in pipe = equivalent rectangular section

3) Calculate the energy and Froude number of the flow entering the dissipator:

   $$H_0 = y_e + \frac{v_0^2}{2g}$$

   $$Fr = \frac{v_0}{\sqrt{g * y_e}}$$

4) Determine $H_0/W_B$ by using Figure 8-2.
5) Calculate the required width of the basin (rounding up to the nearest foot):

   $W = H_0/(H_0/W_B)$

6) Obtain the remaining dimensions from Table 8-3 and Table 8-4.

#### 8.3.4.2 Design Example: USBR Type VI Impact Basin

Given: Pipe entering dissipator with:

$Q = 350$ cfs

$V_0 = 30$ fps

Calculate $A = Q/V_0$

$A = 350 / 30$

$A = 11.67$ sf

Calculate $y_e = (A/2)^{0.5}$

$y_e = (11.67/2)^{0.5}$

$y_e = 2.42$ ft

Calculate $H_0 = y_e + V_0^2/(2g)$

$H_0 = 2.42 + 30^2/(2 * 32.2)$

$H_0 = 16.40$ ft

$$Fr = \frac{v_0}{\sqrt{g * y_e}}$$

$$Fr = \frac{30}{\sqrt{32.2 * 2.42}}$$

$Fr = 3.4$

$(H_0/W_B) = 1.0$ (Using Fr = 3.4 and Figure 8-2)

$W_B = H_0/(H_0/W_B)$

$W_B = 16.4$ ft / 1.0 = 16.4 ft (round up to 17 ft)

Use Table 8-3 and Table 8-4 from HEC-14 to determine remaining dimensions.

| $W_B$ | $h_1$ | $h_2$ | $h_3$ | $h_4$ | L | $L_1$ | $L_2$ |
|---|---|---|---|---|---|---|---|
| 4 | 3.08 | 1.50 | 0.67 | 1.67 | 5.42 | 2.33 | 3.08 |
| 5 | 3.83 | 1.92 | 0.83 | 2.08 | 6.67 | 2.92 | 3.83 |
| 6 | 4.58 | 2.25 | 1.00 | 2.50 | 8.00 | 3.42 | 4.58 |
| 7 | 5.42 | 2.58 | 1.17 | 2.92 | 9.42 | 4.00 | 5.42 |
| 8 | 6.17 | 3.00 | 1.33 | 3.33 | 10.67 | 4.58 | 6.17 |
| 9 | 6.92 | 3.42 | 1.50 | 3.75 | 12.00 | 5.17 | 6.92 |
| 10 | 7.58 | 3.75 | 1.67 | 4.17 | 13.42 | 5.75 | 7.67 |
| 11 | 8.42 | 4.17 | 1.83 | 4.58 | 14.58 | 6.33 | 8.42 |
| 12 | 9.17 | 4.50 | 2.00 | 5.00 | 16.00 | 6.83 | 9.17 |
| 13 | 10.17 | 4.92 | 2.17 | 5.42 | 17.33 | 7.42 | 10.00 |
| 14 | 10.75 | 5.25 | 2.33 | 5.83 | 18.67 | 8.00 | 10.75 |
| 15 | 11.50 | 5.58 | 2.50 | 6.25 | 20.00 | 8.50 | 11.50 |
| 16 | 12.25 | 6.00 | 2.67 | 6.67 | 21.33 | 9.08 | 12.25 |
| 17 | 13.00 | 6.33 | 2.83 | 7.08 | 21.50 | 9.67 | 13.00 |
| 18 | 13.75 | 6.67 | 3.00 | 7.50 | 23.92 | 10.25 | 13.75 |
| 19 | 14.58 | 7.08 | 3.17 | 7.92 | 25.33 | 10.83 | 14.58 |
| 20 | 15.33 | 7.50 | 3.33 | 8.33 | 26.58 | 11.42 | 15.33 |

**Table 8-3: Baffle Wall Dissipator Dimensions in Feet USBR Type VI Impact Basin (Thompson & Kilgore, 2006, Table 9.2)**

| $W_B$ | $W_1$ | $W_2$ | $t_1$ | $t_2$ | $t_3$ | $t_4$ | $t_5$ |
|---|---|---|---|---|---|---|---|
| 4 | 0.33 | 1.08 | 0.50 | 0.50 | 0.50 | 0.50 | 0.25 |
| 5 | 0.42 | 1.42 | 0.50 | 0.50 | 0.50 | 0.50 | 0.25 |
| 6 | 0.50 | 1.67 | 0.50 | 0.50 | 0.50 | 0.50 | 0.25 |
| 7 | 0.50 | 1.92 | 0.50 | 0.50 | 0.50 | 0.50 | 0.25 |
| 8 | 0.58 | 2.17 | 0.50 | 0.58 | 0.58 | 0.50 | 0.25 |
| 9 | 0.67 | 2.50 | 0.58 | 0.58 | 0.67 | 0.58 | 0.25 |
| 10 | 0.75 | 2.75 | 0.67 | 0.67 | 0.75 | 0.67 | 0.25 |
| 11 | 0.83 | 3.00 | 0.67 | 0.75 | 0.75 | 0.67 | 0.33 |
| 12 | 0.92 | 3.00 | 0.67 | 0.83 | 0.83 | 0.75 | 0.33 |
| 13 | 1.00 | 3.00 | 0.67 | 0.92 | 0.83 | 0.83 | 0.33 |
| 14 | 1.08 | 3.00 | 0.67 | 1.00 | 0.92 | 0.92 | 0.42 |
| 15 | 1.17 | 3.00 | 0.67 | 1.00 | 1.00 | 1.00 | 0.42 |
| 16 | 1.25 | 3.00 | 0.75 | 1.00 | 1.00 | 1.00 | 0.50 |
| 17 | 1.33 | 3.00 | 0.75 | 1.08 | 1.00 | 1.00 | 0.50 |
| 18 | 1.33 | 3.00 | 0.75 | 1.08 | 1.08 | 1.08 | 0.58 |
| 19 | 1.42 | 3.00 | 0.83 | 1.17 | 1.08 | 1.08 | 0.58 |
| 20 | 1.50 | 3.00 | 0.83 | 1.17 | 1.17 | 1.17 | 0.67 |

**Table 8-4: Baffle Wall Dissipator Dimensions in Feet USBR Type VI Impact Basin (Thompson & Kilgore, 2006, Table 9.2)**

### 8.3.5 Velocities Greater than 30 fps and or Discharges Greater than 400 cfs

Where velocities exceed 30 fps, discharges exceed 400 cfs, or debris build-up may cause substantial clogging, a special design is required. Engineering Monograph No. 25 by the U. S. Bureau of Reclamation (Peterka, 1984) as updated by the Federal Highway Administration Hydraulic Engineering Circular Number 14 (Thompson & Kilgore, 2006) should be used.

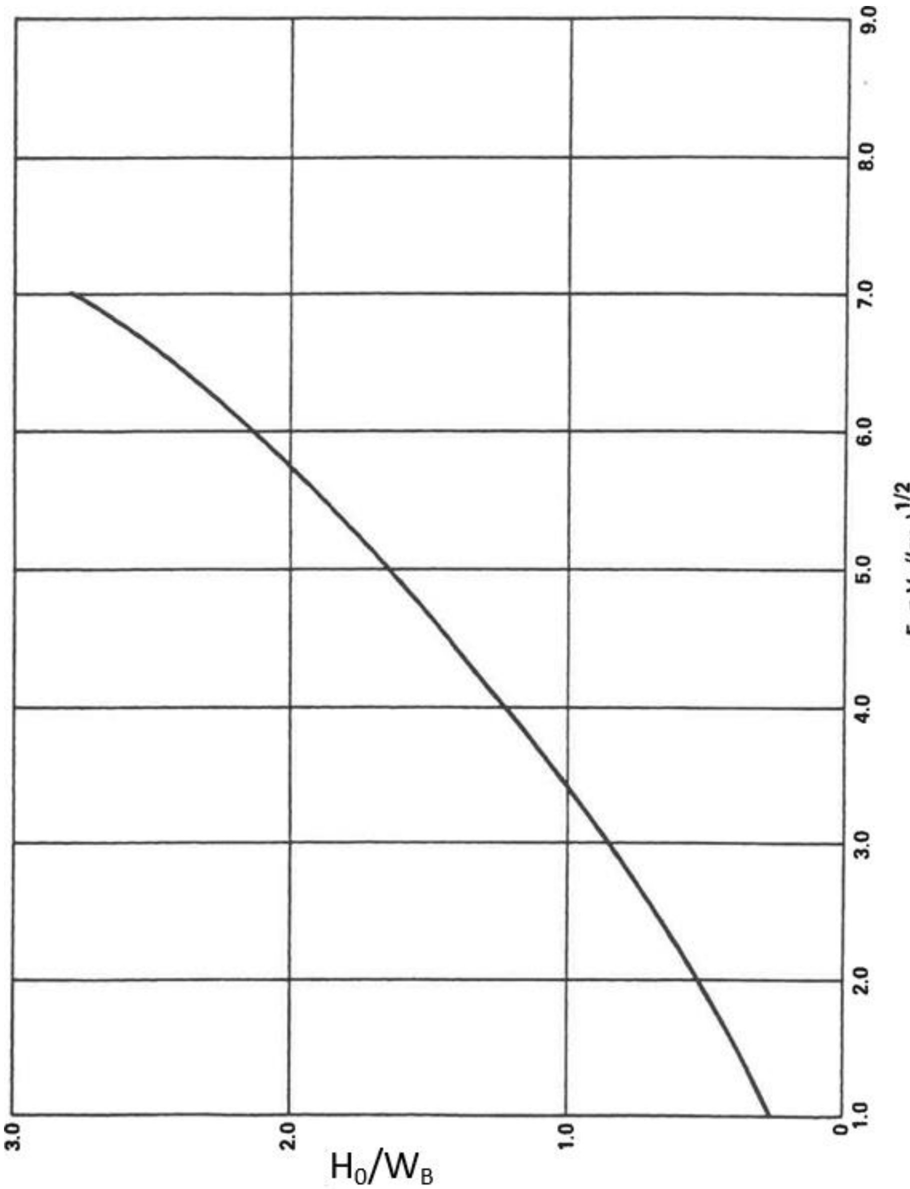



**Figure 8-2: Basin Wall Dissipator Based on Froude Number for USBR Type VI Impact Basin**

## 8.4 References

Brown, S.A., Schall, J.D., Morris, J.L, Doherty, C.L., Stein, S.M., & Warner, J.C. (2009). *Urban Drainage Design Manual: Hydraulic Engineering Circular No. 22 (HEC 22), Third Edition*. Federal Highway Administration. FHWA-NHI-10-009, September.

Thompson, P. L., & Kilgore, R. T. (2006). *Hydraulic Design of Energy Dissipators for Culverts and Channels: Hydraulic Engineering Circular Number 14 (HEC 14), Third Edition*. Federal Highway Administration. FHWA-NHI-06-086, July.

Peterka, A. J. (1984). *Hydraulic Design of Stilling Basins and Energy Dissipators*. Engineering Monograph No. 25. U.S. Bureau of Reclamation.

Public Works Standards, Inc. (2009). *Standard Plan Public Works Construction (2009 Edition, Standard Plan 383-2)*. NBi Building News.

# Chapter 9 Debris Facilities

## 9.1 Introduction

This chapter discusses the design criteria and standard features for debris and erosion facilities within the County of Orange. Debris facilities discussed include debris racks, debris posts, permanent desilting basins, and debris basins.

## 9.2 Design Criteria

Facilities with tributary areas consisting of natural cover are subject to potential for silt or debris flows. Debris or silt flows have the potential to clog downstream facilities. There are several ways to accommodate for debris or silt flow including flow bulking, debris barriers, desilting basins or debris basins. The type of debris protection required depends on the type of debris expected. Fine silts and clays may require flow bulking or permanent desilting basins, while coarse debris or heavily vegetated tributary areas are better suited for debris barriers and debris basins. For debris classification the most recent editions of HEC 9 from FHWA (Bradley, J. B, et al.,2005) should be consulted. The type of debris facility should be discussed with OCPW before commencing design. OC Watersheds staff shall be consulted on water quality issues.

Debris facilities require periodic maintenance to assure proper function. All debris facilities must have an Operation and Maintenance Plan (O&M Plan). The plan must be fully permitted by the resource agencies if O&M activities impact environmental-regulators jurisdictional areas or if species of concern are also impacted. The plan must detail the regular inspection and maintenance including the time interval and/or maintenance indicators. The O&M plan shall assure that the vegetation and debris are removed or maintained on a regular basis to maintain the functionality of the system.

The project owner and design engineer shall consult with OCPW to determine the appropriate maintenance mechanism required for a project. At a minimum, privately owned and maintained debris facilities shall have a recorded easement agreement with a covenant binding on successors or other mechanism acceptable to the governing Agency. Typically, the easement will cover the debris basin or an area around the debris barrier to provide adequate access and maintenance.

## 9.3 Bulking Factor

In areas prone to high sediment and debris concentrations, the use of a bulking factor ($F_b$) can help provide for adequately sized facilities. Bulking has been defined as increasing the clear-water discharge to account for high concentrations of sediment in the flow. Mud and debris flows, which can significantly increase the volume of flow transported from a watershed, most often occur in mountainous or foothill areas subject to wildfires with subsequent soil erosion, and in arid regions near alluvial fans and other zones of geomorphic and geologic activity.

The bulking method used herein is for burned watersheds taken from the Los Angeles County regression curves (also known as the Debris Method presented by the US Army Corps of Engineers in 1992) (USACE, 2000). The method is intended for mud and debris flows from areas subject to fires and subsequent erosion during design storm events. The method may predict overly conservative bulking factors for the design of bridges, culverts, and other infrastructure.

$$F_b = 1 + \left[\frac{D_y}{120{,}000}\right]$$

Where:

$D_Y$ = Design storm debris production rate for the study watershed in cubic yards per square mile ($yd^3/mi^2$)

For Areas = $0.1\ to\ 3.0\ mi^2$

$$Log\ D_y = 0.65(Log\ P) + 0.62(Log\ RR) + 0.18(Log\ A) + 0.12(FF)$$

Where:

$D_Y$ = Unit Debris Yield ($yd^3/mi^2$)

$P$ = Maximum 1-hour Precipitation (inches, taken to 2 decimal places after the decimal point, times 100)

$RR$ = Relief ratio calculation of the difference in elevation (feet) between the highest point in the watershed (measured at the end of the longest stream) and the lowest point (at the debris collection site) and dividing the difference between these two by the maximum stream length (in miles) as measured along the longest stream (ft/mi)

$A$ = Drainage Area ($ac$)

$FF$ = Non-dimensional Fire Factor (Figure 2, USACE LA District, Debris Method 2000 or latest edition)

In lieu of using the simplified method presented here, a bulking factor study can be conducted by a California Registered Engineer and submitted to OCPW for review and approval.

## 9.4 Hydraulic Design of Debris Basins and Barriers

Care should be taken to ensure that natural or earthen facilities downstream of debris facilities do not experience increased erosion due to removal of sediment from the stream flow. In addition, the effect of sediment removal on beach sand replenishment shall be considered.

### 9.4.1 Debris Racks

Debris racks provide a physical barrier across the upstream face of channels or culverts. Debris racks vary greatly in size and materials. Many factors influence the design of debris racks, including: (1) the size and type of debris; (2) the size of culvert or structure being protected; (3) the amount of flow; and (4) flooding issues resulting from the device.

Debris racks shall typically have an open area equivalent to approximately four times the flow area of the conduit or channel they are protecting to maintain flow conveyance and reduce head loss. The height of the rack shall typically extend above the expected depth of flow under the design storm. The design engineer shall consider the use of sloped racks to reduce the risk of pinning debris where applicable. Debris racks shall be well secured but removable for the purposes of maintenance. Standard Plan No. 361-2 from the Standard Plans for Public Works Construction or “Green book” (Public Works Standards, Inc., 2009) offers details for a typical inclined trash rack; however, bars should be ½” x 3” minimum.

Grate length should be limited to 13 feet before the use of a center I-Beam support is required. If an I-Beam support is used, the grate should be constructed in 2 parts joined at the beam. If the grate is wider than the standard plan, then a center support that bolts to the invert of the inlet structure and connects to the I-Beam is required. OCPW requires horizontal flow diverter bars (1/2’x 3”x 5 ½” steel bars) at approximately 4 points (or 3’ to 4’ minimum) along the length of the grate. The flow diverter bars should be placed such that the downstream end is higher and is at a 6° upward angle to level the flow. The bars should be placed on every other opening (except when needed for support, then they should be continuous). The 6° angle was developed empirically and works well for creating a hydraulic jump that forces the debris up and allows the flow to pass freely. Maintenance access to the debris rack is critical and must be provided. Access can take the form of an easement, public trail, or road suitable to OCPW. Finally, the design engineer shall specify debris racks only when special circumstances warrant their use.

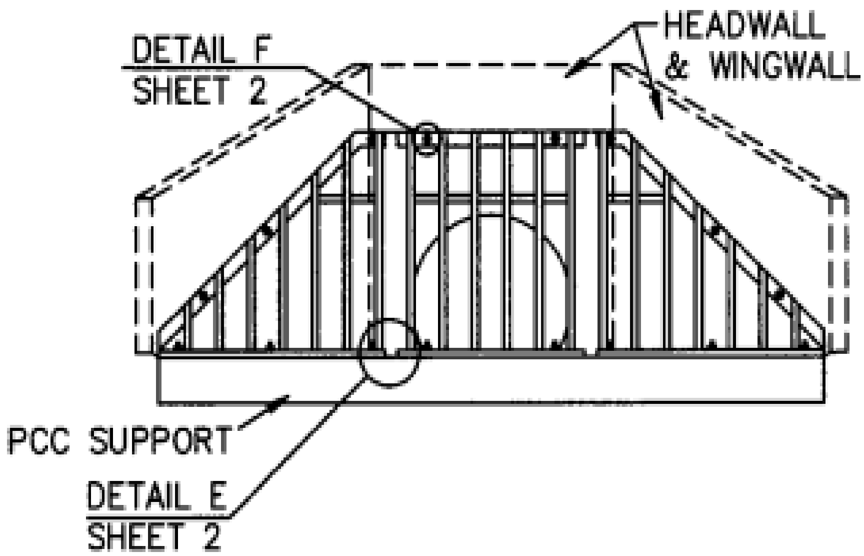



**Figure 9-1: Trash Rack (Inclined) Standard Plan 361-2 (Public Works Standards, Inc., 2009)**

### 9.4.2 Debris Posts

A debris post is a structural system of posts placed upstream of a culvert entrance causing debris to deposit before entering the culvert. The design engineer shall specify debris posts only when special circumstances warrant their use.

The posts shall be a minimum of 4 inches in diameter and are usually constructed of metal embedded in a concrete base. Posts are typically spaced at 1/3 of the culvert diameter to a maximum of 24 inches and are placed upstream of the culvert entrance a distance of twice the culvert diameter where practicable.

The posts shall be designed assuming the barrier to be 100 percent effective in blocking the flow; the barrier will therefore act as a submerged sharp-crested weir with a height equivalent to the height of the debris posts. The design engineer must check that the water elevation spilling over the top of the weir will continue to flow towards the culvert and not flood the surrounding area.

Debris posts shall be embedded to a depth adequate to resist hydrostatic pressures and help prevent failures due to scour. The Los Angeles County Flood Control District *Debris Dams and Basins Design Manual* (1979) provides an equation for embedment depth that may be used.

$$L = (1.85) * \sqrt[3]{\frac{M_o}{R}}$$

$$M_o = \frac{M}{d}$$

Where:

| | | |
|---|---|---|
| $L$ | = | Length of Embedment (ft) |
| $R$ | = | 300 $(\frac{(lb/ft^2)}{ft})$ |
| $M$ | = | Moment applied to barrier (ft-lb) |
| $d$ | = | Diameter of pipe encasement (ft) |

The design engineer shall compare the embedment depth to the potential local scour depth near the debris posts and culvert entrance and specify an appropriate embedment depth based on these values.

Debris production Categories 3 and 4 per the OCFCD Design Manual will require debris posts. Other areas where debris can be a factor, based on OCPW review, may also require debris posts.

### 9.4.3 Permanent Desilting Basins

Permanent desilting basins are typically used to prevent siltation of downstream culverts or channels. The design engineer shall specify permanent desilting basins only when special circumstances warrant their use. The design engineer shall confer with OCPW to determine any requirements.

Temporary desilting basins used to control sediment generated during construction activities have special design requirements that are not addressed in this manual. For information on designing temporary construction desilting basins, the design engineer is encouraged to review the most recent State Water Resources Control Board's Construction General Permit which addresses storm water discharges associated with construction activities and the governing agency's storm water standards.

The design of permanent desilting basins to prevent downstream sediment deposition from sources other than construction activities relies on a number of factors. The design engineer shall consider maintenance requirements during design. Vehicular access for maintenance is a requirement for all basins (access can be in the form of a public road, easement, or other mechanism suitable to OCPW). The design engineer is encouraged to consider the remoteness of a particular basin; this factor may make a larger basin requiring less frequent maintenance more practical than a smaller basin requiring maintenance that is more frequent.

The design engineer shall submit calculations demonstrating the inflow of sediment to the basin. Methods that may be used to calculate sediment volumes, are the Flaxman Method, U.S. Army Corps of Engineers Los Angeles District Method for the Prediction of Debris Yield (2000), or the Modified Universal Soil Loss Equation (MUSLE). The design engineer shall obtain agency approval before using other methods.

Permanent desilting basins must drain within 72 hours and have a spillway capable of conveying the peak design flow without overtopping the basin. Desilting basins falling under State jurisdiction (see Chapter 7, "Detention Basin Design") are typically prohibited. The design calculations shall include the maximum allowable sediment level within the basin. All flood routing and outlet calculations shall be performed according to Chapter 7, "Detention Basin Design".

### 9.4.4 Debris Basins

The use of debris basins is sometimes necessary to capture heavy debris loads, including large boulders. Typically, debris basins serve regional areas as opposed to specific developments or lots. While the design of debris basins is similar to that of desilting and detention basins, the larger debris volumes and the potential for large rocks increase the importance of proper design. A detailed discussion of predicting debris yield and the design of debris basins is outside the scope of this manual. The design engineer shall use both of the following references: Corps of Engineers' Los Angeles District Method for Prediction of Debris Yield (2000) and the Los Angeles County Department of Public Works' Sedimentation Manual 2nd Edition (March 2006). Because of the potential for debris basins to affect numerous properties, the design engineer shall contact the OCPW before beginning design to coordinate the design criteria, submittal requirements, and any legal/regulatory issues.

Retarding basin proposals for flow-through facilities shall include sediment and debris analyses, and the design of the retarding basin shall provide sufficient additional volume to store the sediment and debris expected to be deposited in the basin. Bypass channel low flows shall be confined to an engineered pilot channel or vee to facilitate sediment movement.

The additional basin-volume designed to provide for the accommodation of sediment and debris shall be considered as dead storage (not an active basin volume) for routing of the design flood through the basin.

Flow through basins design shall include:

1. The 100-year sediment production of the existing and ultimate land uses for the watershed (i.e., sediment delivered to the basin from a one time, 100-year storm event) shall be calculated using both methods discussed above (four scenarios total) and compared. The method resulting in the highest accumulation shall have its resulting sediment volume added as dead storage to the basin.
2. For flow-through basins designed to accept runoff from Debris Category three (3) and four (4) catchments as defined and listed in the OCFCD Design Manual (or as determined by the reviewer/engineer), suitable debris catching or mitigating structures shall be constructed upstream of basin inlets to minimize debris accumulation within the basin. The channel(s) upstream of the basin and basin inlet structures shall be designed to accommodate debris accumulation and flow.
3. Where construction of suitable debris mitigating structures is impracticable, discussion with OCPW shall be required. Basin inlets shall be sized to allow for debris laden flow, and a 20% increase in the calculated sediment yield determined in steps one (1) and two (2) above shall be added as dead (non-routed) storage to the total basin volume.
4. For basin designs in highly urbanized areas where debris and sediment may not be an issue, the 20% criterion may be reduced with approval of the District's Chief Engineer.
5. Basin outlet facilities shall be designed to account for debris and sediment. Basin outlet(s) shall be sized to pass all debris and sediment, or protective grates or bars shall be designed (both structurally and hydraulically) to guard from the possibility of clogging or choking the outlet structure.
6. The hydraulic capacity (cross sectional area of the openings) of all trash racks shall be a minimum of 200% of the hydraulic capacity of the outlet structure conduit to allow for the possibility of clogging and plugging.
7. Where grates and bars cover basin outlet(s), grate and bar openings shall not be less than four (4) inches. Basin storage shall be computed assuming a debris induced reduction in total grate or

bar outlet opening size computed from an approved engineering analysis and all available manufacturer's literature.

8. Basin outlet(s) shall be designed above sediment accumulation elevations, or the basin and outlet(s) shall be designed to account for complete (100%) outlet clogging up to the total computed sediment storage elevation. Alternatively, the basin shall be designed to eliminate sediment and debris from clogging the basin outlet(s) such as the use of a properly designed debris pool.
9. Conditions where small channel choke-boxes or small culvert cells/conduits can completely clog or seal from debris accumulation shall be avoided.

Flow-by basins shall account for debris and sediment where piers, stop logs, choke boxes, channel transitions, etc. exist to restrict downstream flows in the bypass channel. The design engineer shall analyze the bypass channel and side weir spillway for both the condition of debris accumulation and no debris accumulation. Probable debris accumulation occurs for higher discharges into the basin and results in greater basin storage volume.

Combination type basins facilities that have both flow-though and flow-by features shall account for debris and sediment as both a flow-through and a flow-by basin, not as one or the other.

## 9.5 References

American Public Works Association and Associated General Contractors of Southern California (APWA-AGC). (2012). *California Standard Specifications for Public Works Construction (12th Edition)*.

Bradley, J. B., Richards, D. L., & Bahner, C. D. (2005). *Debris Control Structures—Evaluation and Countermeasures: Hydraulic Engineering Circular 9 (Third Edition).* Federal Highway Administration.

County of San Diego Department of Public Works Flood Control Section. (2005). *Drainage Design Manual*. http://www.sandiegocounty.gov/dpw/floodcontrol/floodcontrolpdf/drainage-designmanual05.pdf

Los Angeles County Department of Public Works. (2006). *Sedimentation Manual (2nd Edition)*.

Los Angeles County Flood Control District. (October 1979). *Debris Dams and Basins Design Manual*.

Orange County Public Works. (2018, September). *Standard Plans (2018 edition)*.

Public Works Standards, Inc. (2009). *Standard Plan Public Works Construction (2009 Edition)*. NBi Building News.

U.S. Army Corps of Engineers, Los Angeles District. (2000). *Los Angeles District Method for Prediction of Debris Yield*.

# Chapter 10 Structures

## 10.1 Introduction

This chapter discusses the structural design requirements for local drainage facilities including reinforced concrete pipe and small reinforced concrete boxes. This manual defers to the ACPA for the design of reinforced concrete pipes. Text and figures from ACPA and others are included as is needed to distinguish between their minimum safety factors and the locally applied safety factors that are recommended by this manual.

## 10.2 Design Criteria

The basis for structural design shall be a design life of 100-years for all permanent drainage structures within the County. The most recent editions of the following references shall be used:

- Any hydrology study for the determination of conduit size
- Latest County of Orange Board of Supervisors approved California Building Code
- OCFCD Design Manual – Structures, with addendums or updates
- American Concrete Institute (ACI) codes
- OCPW Standard Plans
- Standard Specifications for Public Works Construction
- AASHTO LRFD Bridge Design Specifications
- American Concrete Pipe Association Concrete Pipe Design Manual

In selection of the structural section, the factors to be considered include hydraulics, debris, maintenance, safety, traffic, R/W, property, economics and aesthetics. Extra concrete strength or conduit thickness is typically needed for high flow velocities.

A guide for the protection of cast-in-place and precast reinforced and unreinforced concrete structures against acid and sulfate exposure conditions is included in Limitations on Use of Concrete by Acidity of Soil and Water Section, 4.4.2.4.

## 10.3 Reinforced Concrete Pipe

The design calculation of pipe strength requires the following:

1. Selection of Type of Installation
2. Selection of Bedding
3. Determination of a Bedding Factor
4. Determination of a Safety Factor
5. Calculation of Earth Load
6. Calculation of Live Load
7. Selection of a Pipe Strength

The following sections describe each of the above 7 steps.

### 10.3.1 Types of Installations

All standard installations of reinforced concrete pipe in this manual use pipe and installation terminology as defined in Figure 10-1 unless further refined by OCPW Standard Plan 1319.

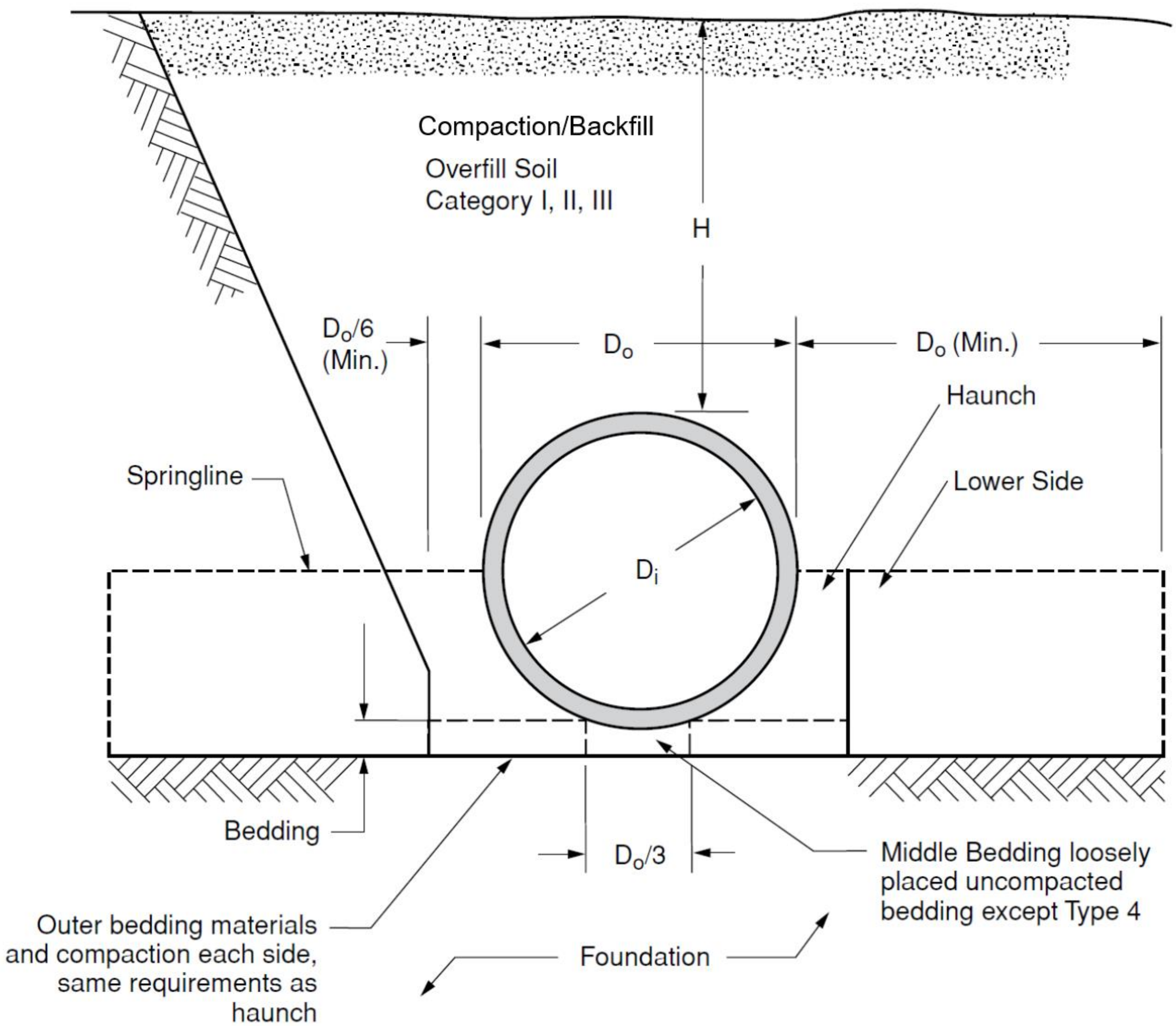



**Figure 10-1: Pipe Installation Terminology (ACPA, 2011, pp 31)**

Definitions for Figure 10-1

$D_i$ = Inside diameter of pipe

$D_o$ = Outside diameter of pipe

H = Cover of fill over pipe

There are four types of pipe installations: Trench, Positive Projecting Embankment, Negative Projecting Embankment, and Jacked or Tunneled. The four installations are defined below and essential features of each installation are shown in Figure 10-2 Types of Pipe Installation.

$B_d = trench\ width\ at\ top\ of\ pipe\ (ft)$
$B_c = outside\ horizontal\ span\ of\ pipe\ (ft)$
$p = projection\ ratio\ for\ positive\ projecting\ and\ induced\ trench\ installations$
$p' = projection\ ratio\ for\ negative\ projecting\ and\ induced\ trench\ installations$

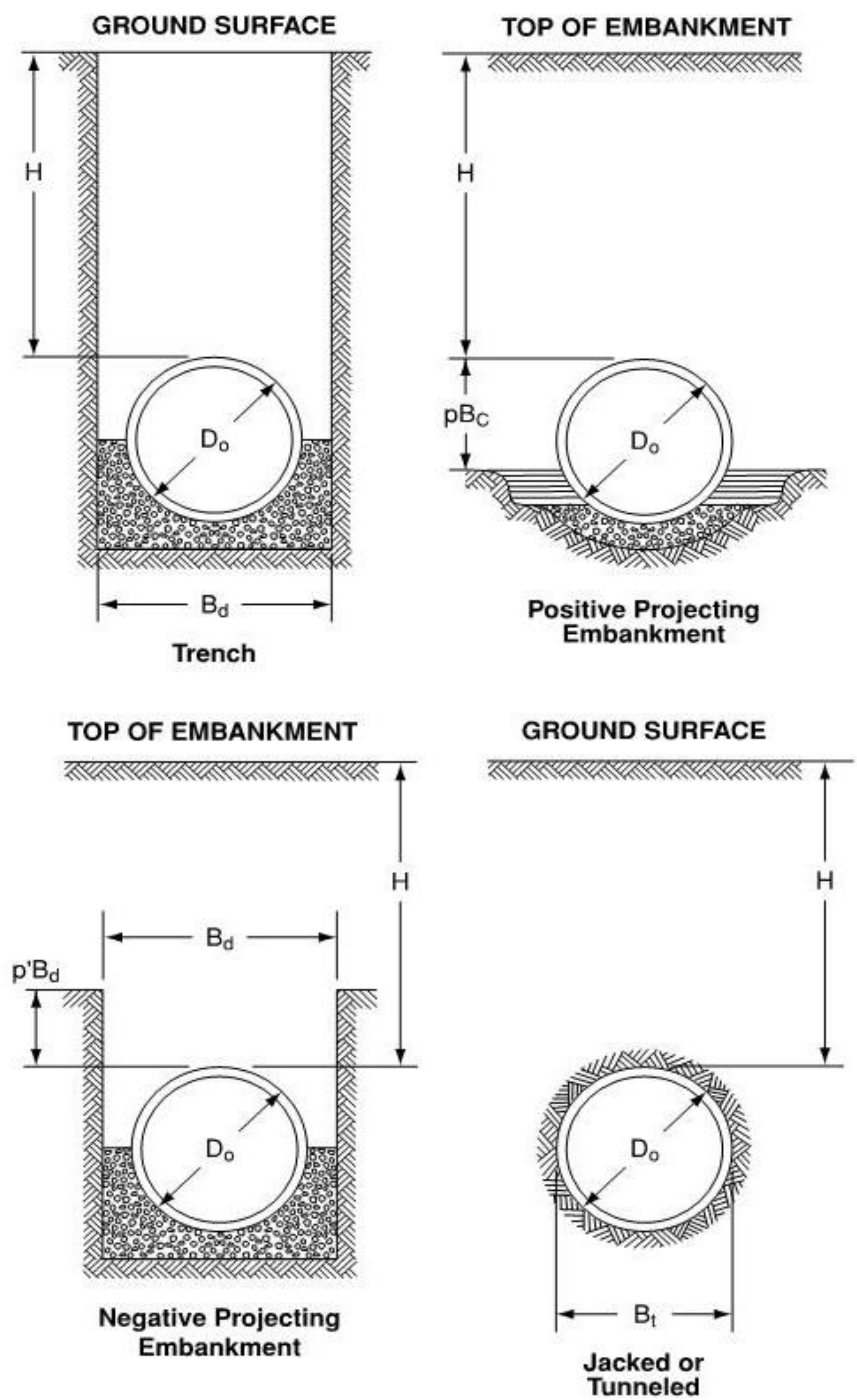



**Figure 10-2: Types of Pipe Installation (ACPA, 2011, pp 27)**

#### 10.3.1.1 Trench

The trench installation is the most common installation for local storm drains in Orange County. In the trench condition, the pipe is installed in a narrow trench that has been excavated in undisturbed soil then covered with backfill to the existing ground.

#### 10.3.1.2 Positive Projecting Embankment

The positive projection embankment installation is used when the storm drain pipe is installed in a relatively flat ground, including streambeds or drainage paths. The pipe is installed on the existing ground or compacted fill and then covered by an earthen fill or embankment.

#### 10.3.1.3 Negative Projecting Embankment Installation

The negative projection embankment installation is used when a culvert is installed in a relatively flat stream bed or drainage path. The pipe is installed in a shallow trench with depth such that the top of the pipe is below the existing ground surface or compacted fill then it is covered with earth fill or embankment which extends above the original ground level.

#### 10.3.1.4 Jacked or Tunneled

The jacked or tunneled installation is used where surface conditions prohibit conventional open excavation and backfill methods, or where it is necessary to install the pipe under an existing utility or embankment. A jacking pit is dug and the pipe is advanced horizontally underground.

#### 10.3.1.5 Standard Installations of Soil and Minimum Compaction Requirements

Standard installation types are adopted from American Concrete Pipe Association's Concrete Pipe Manual (ACPA, 2011).

Three standard pipe installations and backfill provide an optimum range of soil-pipe interaction characteristics and are outlined in Table 10-1. In all standard installations, the middle third area under the pipe is designated as loosely placed, un-compacted material to allow the pipe to settle into the bedding and achieve improved load distribution. The most desirable construction sequence is to place the bedding to grade; install the pipe to grade; compact the bedding outside of the middle-third of the pipe; and then place and compact the haunch area up to the spring line of the pipe. The bedding outside the middle-third of the pipe may be compacted prior to placing the pipe.

| Installation Type | Bedding Thickness | Haunch and Outer Bedding | Lower Side |
|---|---|---|---|
| Type 1 | $D_o$/24 minimum, not less than 3" (75mm). If rock foundation use $D_o$/12 minimum, not less than 6" (150mm). | 95% Category I | 90% Category 1, 95% Category II, or 100% Category III |
| Type 2 | $D_o$/24 minimum, not less than 3" (75mm). If rock foundation use $D_o$/12 minimum, not less than 6" (150mm). | 90% Category 1, 95% Category II | 85% Category I, 90% Category II, or 95% Category III |
| Type 3 | $D_o$/24 minimum, not less than 3" (75mm). If rock foundation use $D_o$/12 minimum, not less than 6" (150mm). | 85% Category I, 90% Category II, or 95% Category III | 85% Category I, 90% Category II, or 95% Category III |

Notes:

(1) Compaction and soil symbols - i.e. "95% Category I"- refers to Category I soil material with minimum standard Proctor compaction of 95%. See Table 10-2 for equivalent modified Proctor values.

(2) Soil in the outer bedding, haunch, and lower side zones, except under the middle 1/3 of the pipe, shall be compacted to at least the same compaction as the majority of soil in the overfill zone.

(3) For trenches, top elevation shall be no lower than 0.1 H below finished grade, or for roadways, its top shall be no lower than an elevation of 1 foot below the bottom of the pavement base material.

(4) For trenches, width shall be wider than shown if required for adequate space to attain the specified compaction in the haunch and bedding zones.

(5) For trench walls that are within 10 degrees of vertical, the compaction or firmness of the soil in the trench walls and lower side zone need not be considered.

(6) For trench walls with greater than 10-degree slopes that consist of embankment, the lower side shall be compacted to at least the same compaction as specified for the soil in the backfill zone.

(7) Sub-trenches

(a) A sub-trench is defined as a trench with its top below finished grade by more than 0.1 H or, for roadways, its top is at an elevation lower than 1ft below the bottom of the pavement base material.

(b) The minimum width of a sub-trench shall be 1.33 D or wider if required for adequate space to attain the specified compaction in the haunch and bedding zones.

(c) For sub-trenches with walls of natural soil, any portion of the lower side zone in the sub-trench wall shall be at least as firm as an equivalent soil placed to the compaction requirements specified for the lower side zone and as firm as the majority of soil in the overfill zone, or shall be removed and replaced with soil compacted to the specified level.

**Table 10-1: Standard Installations Soil and Minimum Compaction Requirements (ACPA, 2011, pp32)**

Soil types used in this manual are from ACPA Manual Standard Installation Direct Design (SIDD) Soil types whose equivalent soil types are given in Table 10-2.

| | **Representative Soil Types** | | **Percent Compaction** | |
|---|---|---|---|---|
| **SIDD Soil** | **USCS,** | **Standard AASHTO** | **Standard Proctor** | **Modified Proctor** |
| Gravelly Sand (Category I) | SW, SP, GW, GP | A1, A3 | 100<br>95<br>90<br>85<br>80<br>61 | 95<br>90<br>85<br>80<br>75<br>59 |
| Sandy Silt (Category II) | GM, SM, ML,<br>Also GC, SC<br>with less than 20%<br>passing #200 sieve | A2, A4 | 100<br>95<br>90<br>85<br>80<br>49 | 95<br>90<br>85<br>80<br>75<br>46 |
| Silty Clay (Category III) | CL, MH, GC, SC | A5, A6 | 100<br>95<br>90<br>85<br>80<br>45 | 90<br>85<br>80<br>75<br>70<br>40 |

**Table 10-2: Equivalent USCS and AASHTO Soil Classifications for SIDD Soil Designations (ACPA. 2011, pp 33)**

The selection of a Standard Installation for a project should be based on an evaluation of the quality of construction and inspection anticipated. A Type 1 Standard Installation requires the highest construction quality and degree of inspection. Required construction quality is reduced for a Type 2 Standard Installation and reduced further for a Type 3 Standard Installation. A Type 4 Standard Installation requires virtually no construction or quality inspection and it is not recommended by OCPW. Consequently, a Type 4 Standard Installation will require a higher strength pipe, and a Type 1 Standard Installation will require a lower strength pipe for the same depth of installation. Type 2 standard installation is consistent with OCPW Standard Plan 1319.

### 10.3.2 Pipe Bedding Requirements

Bedding distributes the reaction from the vertical load on a pipe around the lower exterior surface of the pipe. The best contact between bedding is obtained from granular materials as they will shift to attain contact with the lower exterior surface of the pipe. Ideal load distribution is attained through the use of clean coarse sand, well-rounded pea gravel, or well graded crushed rock.

#### 10.3.2.1 Bedding Factor

Bedding factors relate field installation pipe strength to manufacturer tested pipe strength. Bedding factors relate pipe strength (see Section 10.3.7) from the three-edge bearing strength measured from plant testing to pipe strength with in-place support as installed. The bedding factor is the ratio of the maximum moment achieved in the plant test over the maximum moment in-field for equivalent vertical loading.

Table 10-3 and Table 10-4 show bedding factors for embankment conditions and trench conditions respectively.

| Pipe Diameter | $B_{fe}$ per Standard Installation | | |
|---|---|---|---|
| | Type 1 | Type 2 | Type 3 |
| 12 in. | 4.4 | 3.2 | 2.5 |
| 24 in. | 4.2 | 3.0 | 2.4 |
| 36 in. | 4.0 | 2.9 | 2.3 |
| 72 in. | 3.8 | 2.8 | 2.2 |
| 144 in. | 3.6 | 2.8 | 2.2 |

Notes:
(1) Pipe diameters other than listed in Table 10-3 can be obtained by interpolation.
(2) Bedding factors are based on the soils being placed with the minimum compaction specified in Table 10-1 for each standard installation.

**Table 10-3: Bedding Factors for Embankment Conditions, $B_{fe}$ (ACPA, 2011, pp 51)**

| Standard Installation | Minimum Bedding Factor, $B_{fo}$ |
|---|---|
| Type 1 | 2.3 |
| Type 2 | 1.9 |
| Type 3 | 1.7 |

Notes:
(1 ) Bedding factors are based on the soils being placed with the minimum compaction specified in Table 10-1 for each standard installation.
(2) For pipe installed in trenches dug in previously constructed embankment, the load and bedding factor should be determined as an embankment condition unless the backfill placed over the pipe is of lesser compaction than the embankment.

**Table 10-4: Trench Minimum Bedding Factors (ACPA, 2011, pp 52)**

For pipe installed with 6.5 ft or less of overfill and subjected to truck loads, the ACPA Concrete Pipe Manual shall be further consulted. Earth load bedding factors for truck live loading should be discussed with OCPW engineers for conformance with manufacturer's recommendations.

### 10.3.2.2 Variable Bedding Factor

As a trench increases in width from the pipe walls it transitions to an embankment condition. The transition width is the trench width where the soil load acts like an embankment condition. Trench transition width tables can be found in the ACPA Concrete Pipe Design Manual (see Appendix D.1 Transition Widths). A variable bedding factor is used to interpolate between the trench condition and the embankment conditon. Figure 10-3 is a diagram for terms used in the variable bedding factor equation.

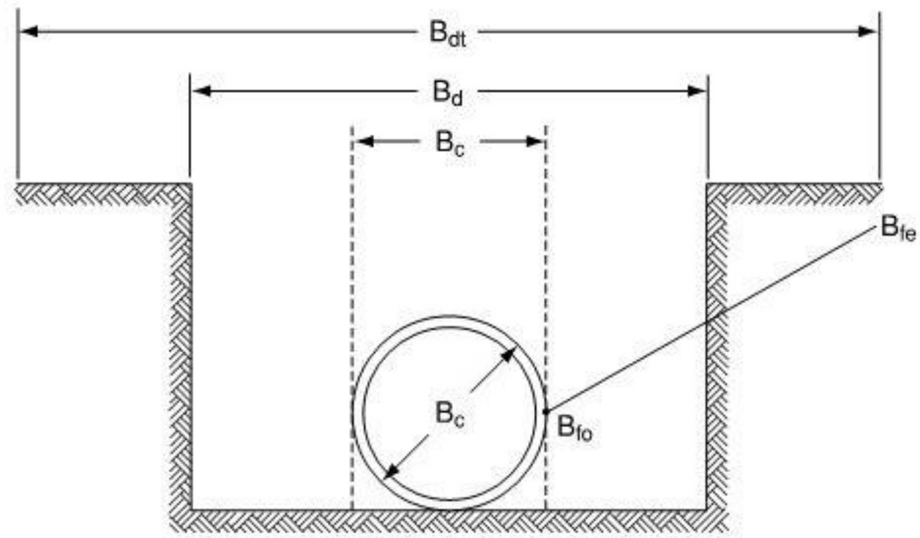



**Figure 10-3: Variable Bedding Factor (ACPA, 2011, pp 52)**

The equation for the variable trench bedding factor is:

$$B_{fv} = \frac{[B_{fe} - B_{fo}][B_d - B_c]}{[B_{dt} - B_c]} + B_{fo}$$

Where:

| | | |
|---|---|---|
| $B_c$ | = | outside horizontal span of pipe (ft) |
| $B_d$ | = | trench width at top of pipe (ft) |
| $B_{dt}$ | = | transition width at top of pipe (ft) |
| $B_{fe}$ | = | bedding factor, embankment |
| $B_{fo}$ | = | minimum bedding factor, trench |
| $B_{fv}$ | = | variable bedding factor, trench |

### 10.3.2.3 Live Load Bedding Factor

Live load bedding factors should be used to account for bedding support for live loads. Exceptions apply if the earth load bedding factor becomes smaller than the live load bedding factor. Refer to the Bedding Factors and Selection of Pipe Strength sections of the ACPA Concrete Pipe Design Manual for live load bedding factor assumptions, applications, and a table of values.

### 10.3.3 Safety Factor

Recommended F.S. = 1.25

### 10.3.4 Earth Loads

The determination of Earth Load is different in trench conditions and embankment condition. These different conditions have significant effects on the loads carried by rigid pipe. In trench conditions, the trench walls help support the soil immediately adjacent to it and relieve the pipe of some of its soil burden. Narrow trench installations are typical but there are situations where the pipe is installed in trenches so wide that they should be treated as an embankment condition. As the trench widens the trench walls do not help support the soil directly above the pipe. The change in load is accounted for by using a Vertical Arching Factor (VAF) multiplied by the load of soil directly above the pipe called prism load (PL). The Vertical Arching Factors for the standard installations are shown in Figure 10-4 and Table 10-5.

#### 10.3.4.1 Positive Projecting Embankment Condition Soil Load

Total load of soil on the pipe is given by the equation:

$$W_E = VAF\ x\ PL$$

Where:

$W_E$ = soil load for positive projecting embankment condition (lb/ft)

VAF = vertical arching factor

PL = prism load (lb/ft)

The prism load, *PL,* is the load of soil directly above pipe and is further defined as:

$$PL = \gamma_S \left[H + \frac{D_O(4-\pi)}{8}\right] D_O$$

Where:

$\gamma_S$ = soil unit weight (lb/ft$^3$)

H = height of fill (ft)

$D_O$ = pipe outside diameter (ft)

The soil unit weight of the proposed backfill is critical for earth load determination.  The soil unit weight in the area of the pipe installation, if used for backfilling can generally be determined from the geotechnical report recommendations.

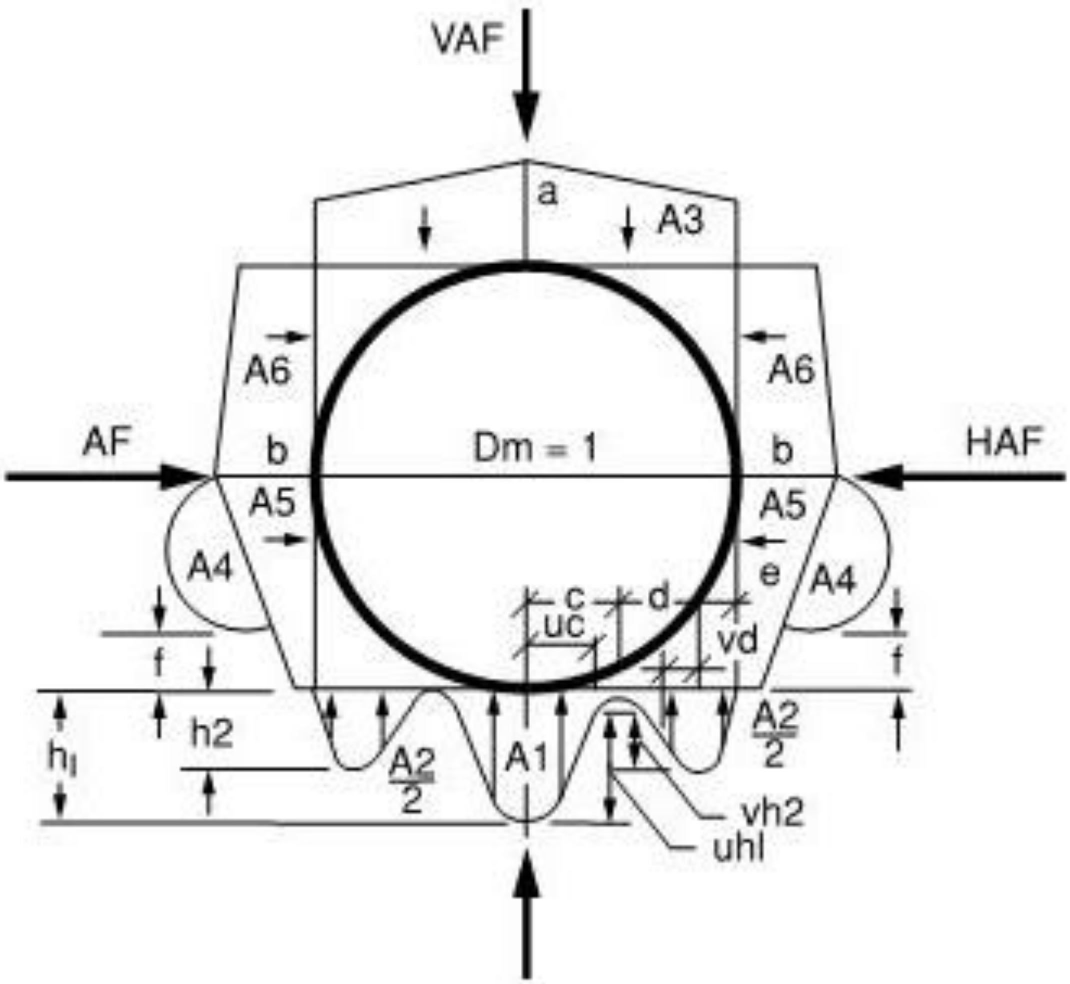



**Figure 10-4: Arching Factors and Coefficients and Heger Earth Pressure Distributions (ACPA, 2011, pp 35)**

### 10.3.4.2 Trench Condition Soil Load

In trench conditions, the existing soil in the trench walls will settle less than the soil backfilled above the pipe. The friction along the trench walls relieve the pipe of some of its soil burden. The Vertical Arching Factors (VAF) given in Table 10-5 will be less.

| Installation Type | VAF | HAF | A1 | A2 | A3 | A4 | A5 | A6 | a | b | c | e | f | u | v |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1.35 | 0.45 | 0.62 | 0.73 | 1.35 | 0.19 | 0.08 | 0.18 | 1.40 | 0.40 | 0.18 | 0.08 | 0.05 | 0.80 | 0.80 |
| 2 | 1.40 | 0.40 | 0.85 | 0.55 | 1.40 | 0.15 | 0.08 | 0.17 | 1.45 | 0.40 | 0.19 | 0.10 | 0.05 | 0.82 | 0.70 |
| 3 | 1.40 | 0.37 | 1.05 | 0.35 | 1.40 | 0.10 | 0.10 | 0.17 | 1.45 | 0.36 | 0.20 | 0.12 | 0.05 | 0.85 | 0.60 |
| 4 | 1.45 | 0.30 | 1.45 | 0.00 | 1.45 | 0.00 | 0.11 | 0.19 | 1.45 | 0.30 | 0.25 | 0.00 | - | 0.90 | - |

Notes:
(1) VAF and HAF are vertical and horizontal arching factors. These coefficients represent non- dimensional total vertical and horizontal loads on the pipe, respectively. The actual total vertical and horizontal loads are (VAF) X (PL) and (HAF) X (PL), respectively, where PL is the prism load.
(2) Coefficients A1 through A6 represent the integration of non-dimensional vertical and horizontal components of soil pressure under the indicated portions of the component pressure diagrams (i.e. the area under the component pressure diagrams).
(3) The pressures are assumed to vary either parabolically or linearly, as shown, with the non-dimensional magnitudes at governing points represented by h1, h2, uh1, vh2, a and b
(4) Non-dimensional horizontal and vertical dimensions of component pressure regions are defined by c, d, e, vc, vd, and f coefficients.
(5) d is calculated as (0.5-c-e).
(6) h1 is calculated as (1.5A1) / (c) (1+u).
(7) h2 is calculated as (1.5A2) / [(d) (1+v) + (2e)]

**Table 10-5: Arching Factors and Coefficients (ACPA, 2011, pp 35)**

Total load of soil on the pipe in a trench condition is given by the equation:

$$W_d = C_d \gamma_S B_d^2 + \frac{D_O^2(4-\pi)}{8}\gamma_S$$

Where:

$W_d$ = soil load for trench condition (lb/ft)
$C_d$ = trench load coefficient
$B_d$ = trench width (ft)
$D_O$ = pipe outside diameter (ft)
$\gamma_S$ = soil unit weight ($lb/ft^3$)

The trench load coefficient, $C_d$, can be calculated from formulas in ACPA 2011 Concrete Pipe Design Manual or read from Figure 10-5.

As trench width increases, the reduction in load from the frictional forces is offset by the increase in soil weight within the trench and pipe loading begins to behave like in an embankment condition. Eventually the embankment condition is reached when the trench walls are too far away from the pipe to help support the soil above the pipe. The transition width where loading conditions transition from trench to embankment conditions, at a particular depth, occurs where the trench load equals the embankment load. Any pipe installed in a trench with width equal to or greater than the transition width should be designed for the embankment conditions. The American Concrete Pipe Association Concrete Pipe Design Manual provides tables with transition widths (Tables 13 -39). A sample is provided as Appendix D.1 showing the different transition widths by pipe size.

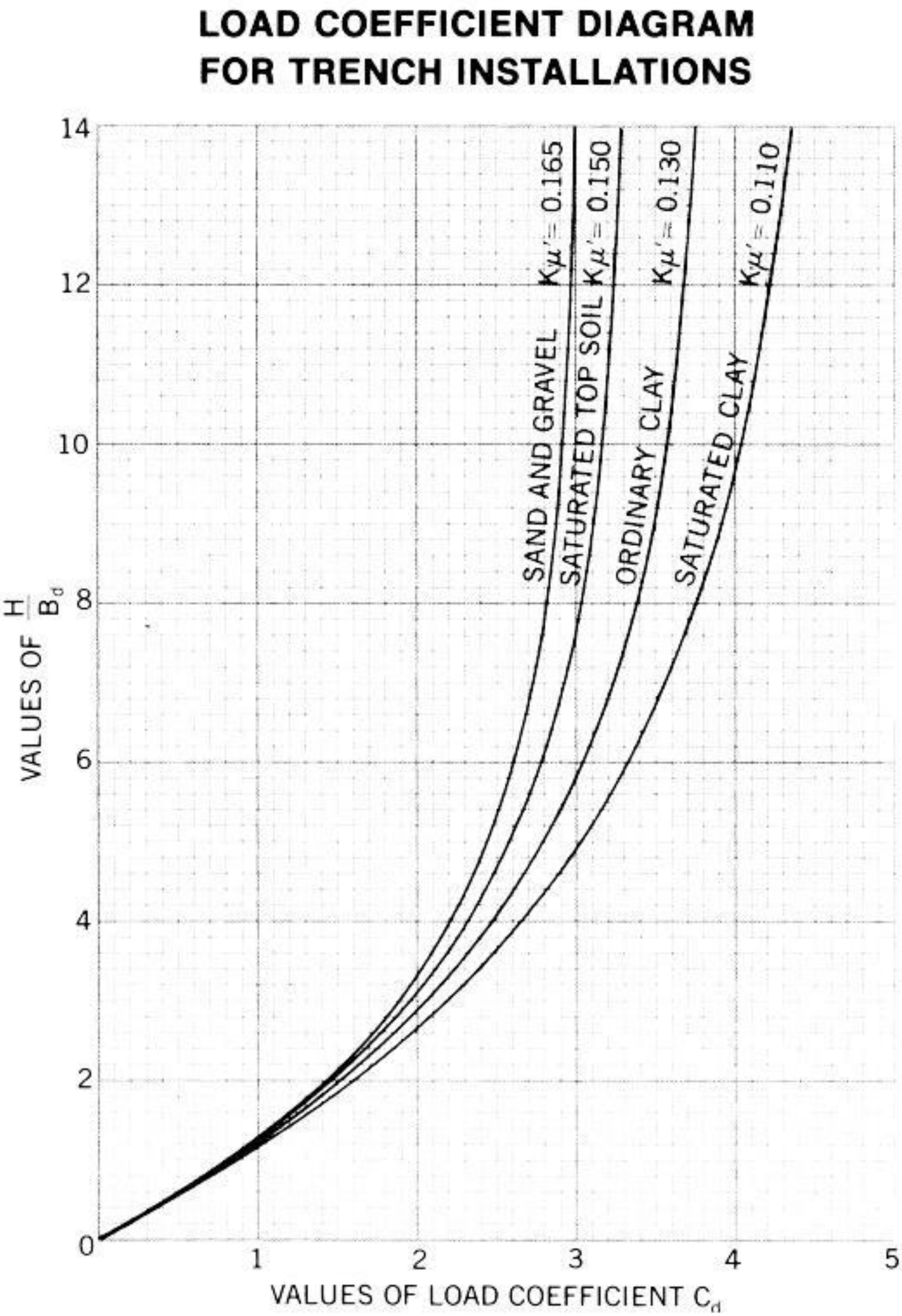



**Figure 10-5: Load Coefficient Diagram for Trench Installations**

### 10.3.4.3 Negative Projection Embankment Condition Soil Load

The soil load on a pipe installed in a negative projecting embankment is computed by the equation:

$$W_n = C_n \gamma_S B_d^2$$

Where:

| | | |
|---|---|---|
| $W_n$ | = | soil load on negative projecting embankment (lb/ft) |
| $C_n$ | = | embankment coefficient |
| $\gamma_S$ | = | soil unit weight, (lb/ft$^3$) |
| $B_d$ | = | trench width (ft) |

The embankment coefficient $C_n$ is further defined in terms of two conditions: 1) for $H \le H_e$ and 2) for $H > H_e$. $H_e$ is the height of fill (ft) from the top of pipe to the plane of equal settlement as determined by a geotechnical report. H is the height of fill.

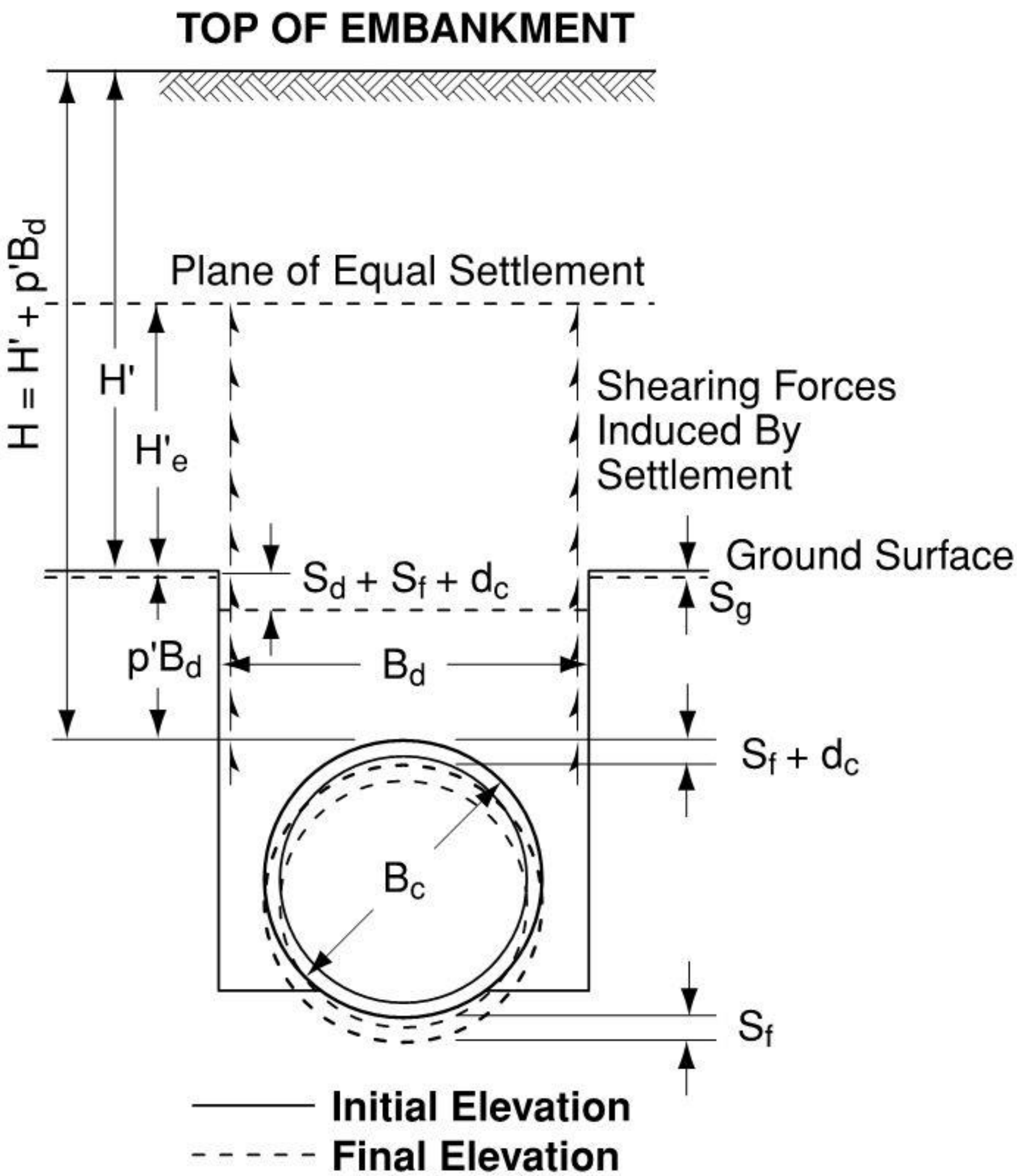



**Figure 10-6: Settlements which Influence Loads on Negative Projection Embankment Installation (ACPA, 2011)**

$d_c$ = deflection of the vertical height of the pipe
$S_g$ = settlement of the natural ground or compacted fill adjacent to the pipe
$S_f$ = settlement of the pipe into its bedding foundation
$S_d$ = compression of the fill material in the trench within the height $p'B_d$ for negative projecting embankment installations

The settlements that influence loads on negative projecting embankment installations are shown in Figure 10-6.

Recommended settlement ratio design values are listed in Appendix D.2. The projection ratio (p’) for this type of installation is the distance from the top of the pipe to the surface of the natural ground or compacted fill at the time of installation divided by the width of the trench. Figures in Appendix D.4 present fill loads in pounds per linear foot for circular pipe based on projection ratios of 0.5, 1.0, 1.5, 2.0 and settlement ratios of 0, -0.1, -0.3, -0.5 and -1.0. The dashed H = p’$B_d$ line represents the limiting condition where the height of fill is at the same elevation as the natural ground surface. The dashed H = $H_e$ line represents the condition where the height of the plane of equal settlement ($H_e$) is equal to the height of fill (H).

### 10.3.4.4 Jacked or Tunneled Condition Soil Load

The earth load on a pipe installed by tunneling is computed by equations found in the ACPA Concrete Pipe Design Manual including:

$$W_t = C_t w B_t^2 - 2cC_t B_t$$

Where:

| | | |
|---|---|---|
| $W_t$ | = | earth load on jacked/tunneled pipe (lb/ft) |
| $B_t$ | = | width of tunnel bore, (ft) |
| $C_t$ | = | jacked or tunneled coefficient, per ACPA 2011 |
| $c$ | = | coefficient of cohesion (psf) |
| $w$ | = | unit weight of soil (lb/ft$^3$) |

In the earth load on a pipe installed by tunneling equation the $C_t w B_t^2$ term is similar to the Negative Projection Embankment equation for soil loads and the $2cC_tB_t$ term accounts for the cohesion of undisturbed soil. Conservative design values of the coefficient of cohesion for various soils are listed in Appendix D.2. Appendix D.3 provides direction to determine the values of the trench load term ($C_t w B_t^2$) in pounds per linear foot for a soil density of 120 pounds per cubic foot and Kμ' values of 0.165, 0.150, 0.130 and 0.110. Appendix D.3 provides references to ACPA tables for earth loads on jacked pipes. These tables can be used to find $2C_tB_t$. This will allow calculation of the cohesion term ($2cC_t B_t$). This can be used to find the total earth load for any given height of cover, width of bore or tunnel, and type of soil. The total earth load is the remainder of the subtraction of the cohesion term from the trench load term.

### 10.3.5 Water Load

Internal water loading need not be always included in the design of buried conduits. In the event of pressurized facilities or high-walled box conduits, inclusion of internal water loading may be required.

$$W_F = \gamma_w \times A$$

Where:

$W_F$ = load on pipe due to fluid within the pipe (lb/ft)

$\gamma_w$ = $62.4\ lb/ft^3$

A = cross sectional area of pipe ($ft^2$)

### 10.3.6 Live Loads

In AASHTO LRFD Bridge Design Specifications vehicular live loading consists of a design truck load or design tandem and a design lane load. Live loading is insignificant at depths to top of pipe greater than 8 ft.

#### 10.3.6.1 HL-93 Design Truck

Figure 10-7 shows the AASHTO design truck dimensions (16.0 kip axles). AASHTO also describes a (alternate) Design Tandem truck that has a pair of 25.0-kip axles spaced 4.0-ft apart also with transverse wheel spacing of 6.0-ft.

#### 10.3.6.2 Wheel Load Area

Wheel load pressure demands are governed by the cover over the pipe, the direction of travel parallel or perpendicular to the pipe, and number of interacting wheels. For direction of travel parallel to the pipe:

- For shallow covers approximately 2 feet the maximum live load pressure is governed by a single dual wheel loading as shown in Figure 10-8. In this case, the live load demand is imposed by 16,000 lbs for 1 set of dual wheels.
- For intermediate depths approximately between 2 feet and 6 feet the maximum live load pressure is given by a single dual wheel from two trucks in passing mode as shown in Figure 10-9. In this case, the live load demand is imposed by 32,000 lbs for 2 sets of dual wheels.
- Larger covers are governed by two single dual wheels of two alternative loads in passing mode as shown in Figure 10-10. In this case, the live load demand is imposed by 50,000 lbs for 4 dual wheels.

For vehicles traveling perpendicular to the pipe, consult ACPA and AASHTO.

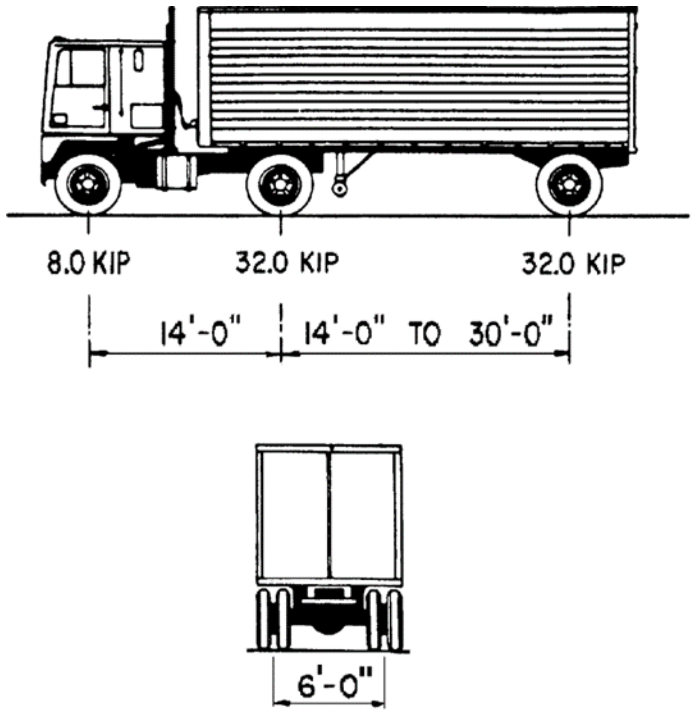



**Figure 10-7: HL-93 Design Truck (Caltrans, 2004)**

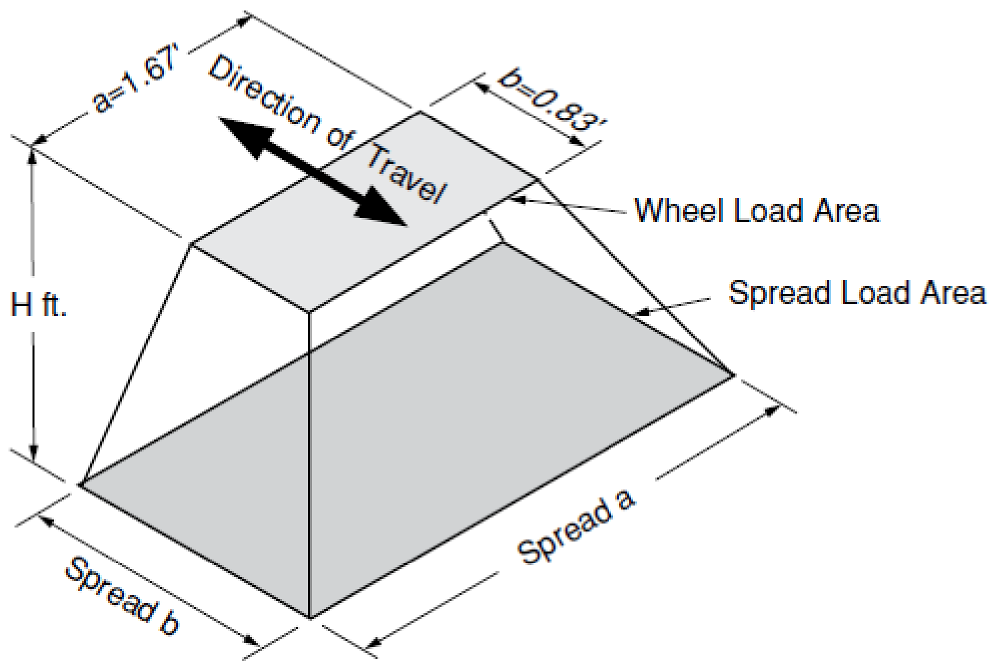



**Figure 10-8: Spread Load Area – Single Dual Wheel (ACPA, 2011, Illustration 4.13)**

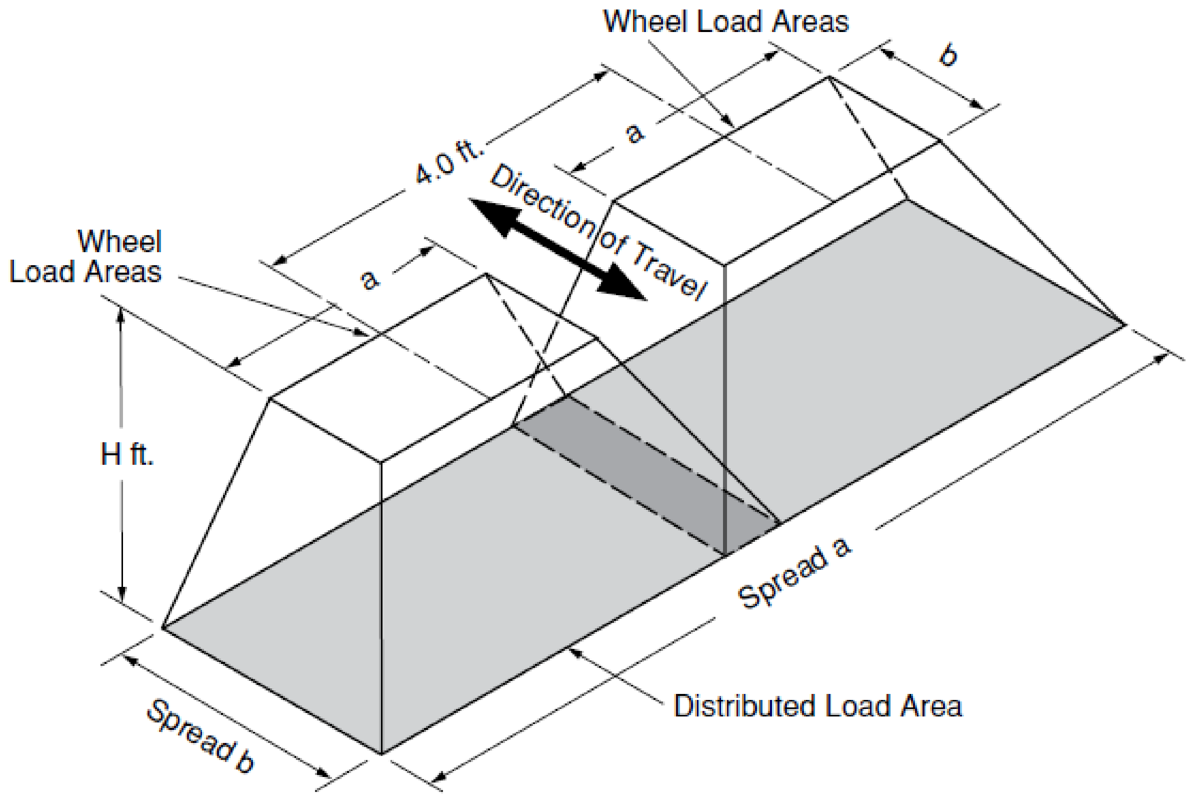



**Figure 10-9: Spread Load Area – Two Single Dual Wheels of Trucks in Passing Mode (ACPA, 2011, Illustration 4.14)**

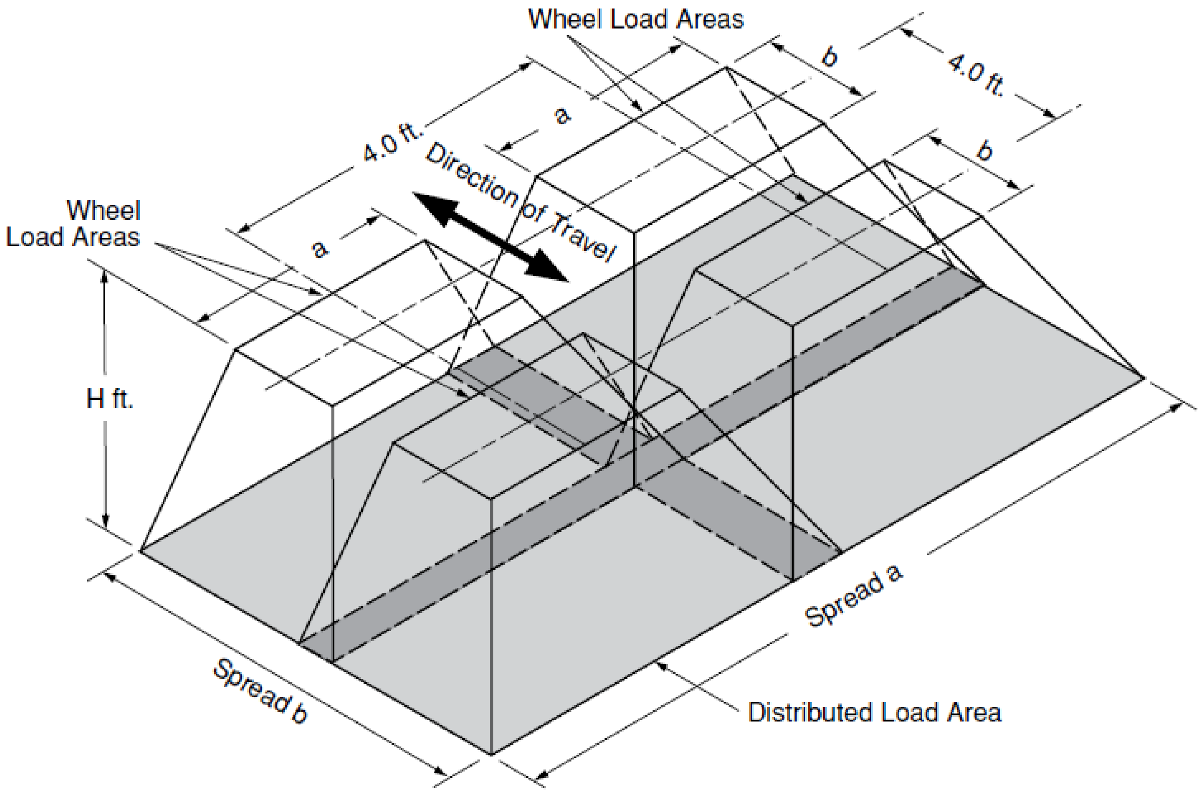



**Figure 10-10: Two Single Dual Wheels of Two Alternative Loads in Passing Mode (ACPA, 2011, Illustration 4.15)**

Unless a designer is certain of pipe orientation live load should be checked for vehicle travel parallel and perpendicular to pipe length.

### 10.3.6.3 Impact Factor / Multiple Presence Factor

Impact Factors for AASHTO LRFD account for the truck load being non-static.

$$IM = \frac{33(1.0 - 0.125H)}{100}$$

Where:

| | | |
|---|---|---|
| *IM* | = | impact factor/dynamic load allowance |
| *H* | = | height of earth cover over the top of the pipe (ft) |

### 10.3.6.4 Average Pressure Intensity

Multiple presence factors are typically a concern for bridges to account for simultaneous loads from multiple trucks. These may need to be included in calculations for the wheel-load average pressure intensity. For single presence, wheel-load average pressure intensity on the subsoil plane at the outside top of the concrete pipe is:

$$w = \frac{P(1 + IM)}{A}$$

Where:

| | | |
|---|---|---|
| $w$ | = | wheel-load average pressure intensity (lb/ft$^2$) |
| $P$ | = | total live load applied at the surface on all interacting wheels (lb) |
| $A$ | = | spread wheel load area at the outside top of the pipe (ft$^2$) |
| $IM$ | = | dynamic load allowance (impact factor) |

### 10.3.6.5 Total Live Load

Maximum possible loads can occur when a truck travels parallel or transverse to the centerline of the pipe. If the orientation of the pipe to the flow of traffic is unknown, the designer should calculate both orientations and select the greater.

$$W_T = (w + L_L)L \times S_L$$

Where:

| | | |
|---|---|---|
| $W_T$ | = | total live load (lbs) |
| $w$ | = | wheel load average pressure intensity (at top of the pipe) (lbs/ft$^2$) |
| $L_L$ | = | lane loading is 64 lbs/ft$^2$ for $0 \leq H < 8$ft, 0 lb/ft$^2$ for $H \geq 8$ ft |
| $L$ | = | dimension of load area parallel to the longitudinal axis of pipe (ft) |
| $S_L$ | = | outside horizontal span of pipe, $B_c$, or dimension of load area transverse to the longitudinal axis of pipe, whichever is less (ft) |

### 10.3.7 Pipe Strength

The required pipe strength is either calculated directly using the D-load equation or it can be looked up directly using Table 10-6: D-load Tables. The pipe strength is dependent on the pipe wall classification which determines its thickness.

#### 10.3.7.1 D-load Calculations

D-load is a numerical classification of pipe strength applied to RCP, which is the result of the following equation:

$$\text{D load} = \left(\frac{W_E + W_F}{B_f} + \frac{W_L}{B_{fLL}}\right) \times FS \ \frac{12}{D}$$

Where:

| | | |
|---|---|---|
| D-load | = | required structural loading capacity (lb/ft$^2$) per ASTM C 76 |
| $W_L$ | = | $W_T/L_e$ (lb/ft) |
| $W_T$ | = | Total Live Load (lb) |
| $L_e$ | = | effective supporting length of pipe (ft) (Refer to ACPA 2011) |
| $W_E$ | = | Earth load (lb/ft) |
| $W_F$ | = | Fluid load (lb/ft) |
| $B_f$ | = | Bedding factor |
| $B_{fLL}$ | = | Live Load bedding factor |
| $D$ | = | Inside diameter (in) |
| $FS$ | = | Factor of safety [FS= 1.25 for OCPW] |

The required D-load shall be calculated after considering live load, earth load, fluid load, bedding and the type of installation. D-loads are expressed in per linear foot per foot of inside diameter.

#### 10.3.7.2 Minimum D-load

The minimum strength of pipe for local streets is 800-D.

The following D-loads Tables may be used versus detailed calculations

### 10.3.7.3 D-load Tables

Assumptions in the D-load tables:

1. D-load ($lb/ft^2$) for Type II Bedding
2. $\gamma_s = 120\ pcf$
3. AASHTO HL-93 live load
4. Positive Projecting Embankment Condition (Conservative Design compared to Trench Condition)
5. FS = 1.25

Indicates need for Type I Installation Bedding
Indicates need for Type II Installation Bedding

| | Fill Height in Feet | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Pipe Size (in)** | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **10** | **11** | **12** | **13** | **14** |
| **12** | 2000 | 2000 | 2000 | 1250 | 1250 | 1000 | 1000 | 1250 | 1250 | 1500 | 1500 | 1500 | 1750 | 1750 |
| **15** | | | | | 1000 | | | | | | | | | |
| **18** | 1750 | 1750 | 1750 | | | | | | | | | | | |
| **21** | | | | | | | | | | | | | | |
| **24** | | | | | | | | | | | | | | |
| **27** | 2250 | | | | | | | | | | | | | |
| **30** | | | | | | | | | | | | | | |
| **33** | 1500 | 1500 | 1500 | | | | | | | | | | | |
| **36** | | | | | | | 1200 | | 1200 | | | | | 1500 |
| **42** | 2200 | 1600 | 1600 | 1100 | | | 1100 | | 900 | | | 1600 | 1300 | 1800 |
| **48** | 1300 | 1300 | 1300 | | | | | | 1000 | 1100 | | | | |
| **54** | 2000 | | | | | | | | | | | | | |
| **60** | 1800 | | | | | | | | | | 1550 | 1300 | | |
| **66** | 1250 | 1250 | 1250 | | 1050 | 1050 | 1050 | | | | | | 1350 | 1850 |
| **72** | | | | | | | 1100 | | | | | | 1400 | 1900 |
| **78** | 1600 | 1000 | 950 | 1000 | | 1000 | 1050 | 1100 | 1350 | 1150 | | | | |
| **84** | | | | | | | | | 1400 | | 1250 | | | |
| **90** | 1400 | | | | | | | 1300 | | | | 1350 | 1450 | 1950 |
| **96** | | | | | | | | | | | | | | 1550 |
| **102** | | | | | 1000 | 1050 | 1100 | | | 1200 | 1300 | | | |
| **108** | | | 1000 | | | | | | | | | 1400 | | |

| | Fill Height in Feet | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Pipe Size (in)** | **15** | **16** | **17** | **18** | **19** | **20** | **21** | **22** | **23** | **24** | **25** | **26** | **27** | **28** |
| **12** | 2000 | 2000 | 2250 | 2250 | 2500 | 2500 | 2500 | 2750 | 2750 | 3000 | 3000 | 3250 | 3250 | 3500 |
| **15** | | | | | 2250 | | | | | | | | | |
| **18** | | | | | | | | | | | | | | |
| **21** | | | | | | | | | | | | | | |
| **24** | | | | | | | | | | | | | | |
| **27** | | | | | | | | | | | | | | |
| **30** | | | | | | | | | | | | | | |
| **33** | | | | | | | | | | | | | | |
| **36** | | | | | | | | | 3000 | | | | | |
| **42** | 1900 | | 2200 | 2300 | 2400 | 3500 | 2600 | 2700 | 2800 | | 3100 | 3200 | 3300 | 3400 |
| **48** | | 2100 | | | | | | | 2900 | | | | | 3500 |
| **54** | 2000 | | | | | | 2700 | 2800 | | | | | | |
| **60** | | | | | | 2600 | | | | | | | | |
| **66** | 1550 | 1650 | 1750 | 2350 | 2450 | 2550 | | | | 3050 | 3150 | 3250 | 3400 | 2750 |
| **72** | | | | | | 2600 | | 2850 | 2950 | | 3200 | 3300 | | 3550 |
| **78** | | 2150 | 2250 | | 2500 | | | | | | | | | |
| **84** | 2050 | | | 2400 | | | 2750 | | | 3100 | | | 3450 | |
| **90** | | | | | | 2650 | | | 3000 | | | | | |
| **96** | | | 2300 | | | | | | | | | | | |
| **102** | | 2200 | | | 2550 | | | 2900 | | | 3250 | 3350 | | 3600 |
| **108** | 2100 | | | 2450 | | | 2800 | | | | | | | |

**Table 10-6: D-load Tables**

Concrete pipes can have three different wall thicknesses per inner diameter size. The thickness of the pipe is classified as Wall A, Wall B, or Wall C. Wall A has the smallest pipe wall thickness and Wall C has the largest. Wall B, as shown in Table 10-11, is normally used in most Orange County Subdivisions. The wall thickness is the primary factor for pipe strength.

| | WALL A | WALL A | WALL B | WALLB | WALL C | WALL C |
|---|---|---|---|---|---|---|
| Internal Diameter (in) | Minimum wall thickness (in) | Approximate Weight (lb/ft) | Minimum wall thickness (in) | Approximate weight (lb/ft) | Minimum wall thickness (in) | Approximate weight (lb/ft) |
| 18 | 2 | 131 | 2 ½ | 168 | ----- | ----- |
| 21 | 2 ¼ | 171 | 2 ¾ | 214 | ----- | ----- |
| 24 | 2 ½ | 217 | 3 | 264 | 3 ¾ | 366 |
| 27 | 2 5/8 | 255 | 3 ¼ | 322 | 4 | 420 |
| 30 | 2 ¾ | 295 | 3 ½ | 384 | 4 ¼ | 476 |
| 33 | 2 7/8 | 366 | 3 ¾ | 451 | 4 ¼ | 552 |
| 36 | 3 | 383 | 4 | 524 | 4 ¾ | 654 |
| 42 | 3 ½ | 520 | 4 ½ | 686 | 5 ¼ | 811 |
| 48 | 4 | 683 | 5 | 867 | 5 ¾ | 1011 |
| 54 | 4 ½ | 864 | 5 ½ | 1068 | 6 ¼ | 1208 |
| 60 | 5 | 1064 | 6 | 1295 | 6 ¾ | 1475 |
| 66 | 5 ½ | 1287 | 6 ½ | 1542 | 7 ¼ | 1735 |
| 72 | 6 | 1532 | 7 | 1811 | 7 ¾ | 2015 |
| 78 | 6 ½ | 1797 | 7 ½ | 2100 | 8 ¼ | 2410 |

**Table 10-7: Dimensions and Approximate Weights of Concrete Pipe**

**(ACPA, 2011, Illustration 5.2 and ASTM C-76 for additional dimensions and weights)**

## 10.4 Reinforced Concrete Box

This section describes the requirements for the structural design of local Reinforced Concrete Boxes (RCBs) with tributary drainage areas less than 640 acres. For RCBs with tributary drainage areas of 640 acres or more, the facility shall be designed per the Orange County Flood Control Design Manual.

### 10.4.1 Concrete

Concrete shall conform to the latest edition Caltrans Bridge Design Specifications (BDS) except that evaluation of $f'_C$ may be based on cores.

### 10.4.2 Reinforcement

Reinforcement shall meet the requirements of Caltrans BDS except that for welded wire fabric a yield strength of 65,000 psi may be used. For wire fabric, the spacing of longitudinal wires shall be a maximum of 8 inches.

### 10.4.3 Design

Design shall conform to applicable sections of the Caltrans BDS except as provided otherwise in this section. For design loads and loading conditions see Caltrans BDS Section 3. Figure 10-11 and Figure 10-12 are provided as a reference. If a box width spans more than 20 feet it must be designed as a bridge. Wheel loads shall be HS20-44 truckloads plus impact (Caltrans, 2004).

#### 10.4.3.1 Distribution of Concentrated Load Effects in Sides and Bottoms

The width of the top slab strip used for distribution of concentrated wheel loads shall also be used for determination of bending moments, shears and thrusts in the sides and bottom.

#### 10.4.3.2 Distribution of Concentrated Loads in Skewed Culverts

Wheel loads on skewed culverts shall be distributed using the same provisions as given for culverts with main reinforcement parallel to traffic.

#### 10.4.3.3 Span Length

When monolithic haunches inclined at 45 degrees are taken into account, negative reinforcement in walls and slabs may be proportioned based on the bending moment where the depth of the haunch equals 1.5 times the thickness of the member.

#### 10.4.3.4 Strength Reduction Factors

Strength reduction factors for load factor design of machine-made boxes may be taken as 1.0 for moment and 0.9 for shear.

#### 10.4.3.5 Crack Control

Structural and geotechnical design trends (including USACE's) recognize three load categories for the modern design concepts as follows:

- Usual
- Unusual
- Extreme

Each of the above categories has different expected performance requirements that is dependent on the frequency of loading condition. Permanent loads are categorized as "Usual". Here, they are further associated with service loads. The service load stress should be computed considering the effects of both bending moment and thrust for crack control computations.

RCBs for local drainage need to comply with Caltrans latest Bridge Design Criteria for crack control. Caltrans standards are considered acceptable for standardized cases that do not require special designs.

As an additional requirement, RCBs for local drainage that are expected to remain continuously wet due to tidal influence are to be designed as Reinforced Concrete Hydraulic Structures. A Load Surcharge factor of 1.3 for resilience purposes shall be applied as is described in the OCFCD-DM. Alternatively, the maximum service load stress in the reinforcing steel for crack control shall be governed by ACI 350, Eq.10-5, as is described in ACI 350-2001 (or later versions).

#### 10.4.3.6 Vees

Vees to concentrate low flows should be placed in the invert slab. Vee depth should be fixed based on a cross-slope of 3/4-inch per foot without regard to the fractions of inches that may result. In multi-barrel sections, the given cross-slope should be projected to the low-flow concentration point nearest the structure centerline. The practice of placing vees in each barrel of multi-barreled structures should be avoided. Fillets shall be included at the top corners of all rigid frame box conduits. They should be either 4" x 4" or 6" x 6" at the contractor's option.

The use of large vees in box conduits having one, three, or other odd numbers of cells introduces the possibility for a reversal of stresses or other adverse effects. With the 3/4-inch per foot cross slope additional structural analysis is not needed.

#### 10.4.3.7 Steel Patterns

Because of the tolerance capabilities of ordinary fabricating shops steel patterns should generally avoid more than one bend per bar. This is especially true where dimensions are critical such as in "U" bars and more than one bend will not be accepted in these cases. Typical steel patterns and other details for single and double box conduits are shown on Caltrans Standard Plans D80 and D81, respectively.

### 10.4.4 Exhibits

The 2015 Caltrans Standard Plans for RCB are shown as examples in Figure 10-12 and Figure 10-13. Refer to Caltrans Standard Plans for most box culverts and OCFCD Design Manual for crossings of regional facilities. The most recent edition of Standard Plans by Caltrans shall be used as reference for concrete box design.

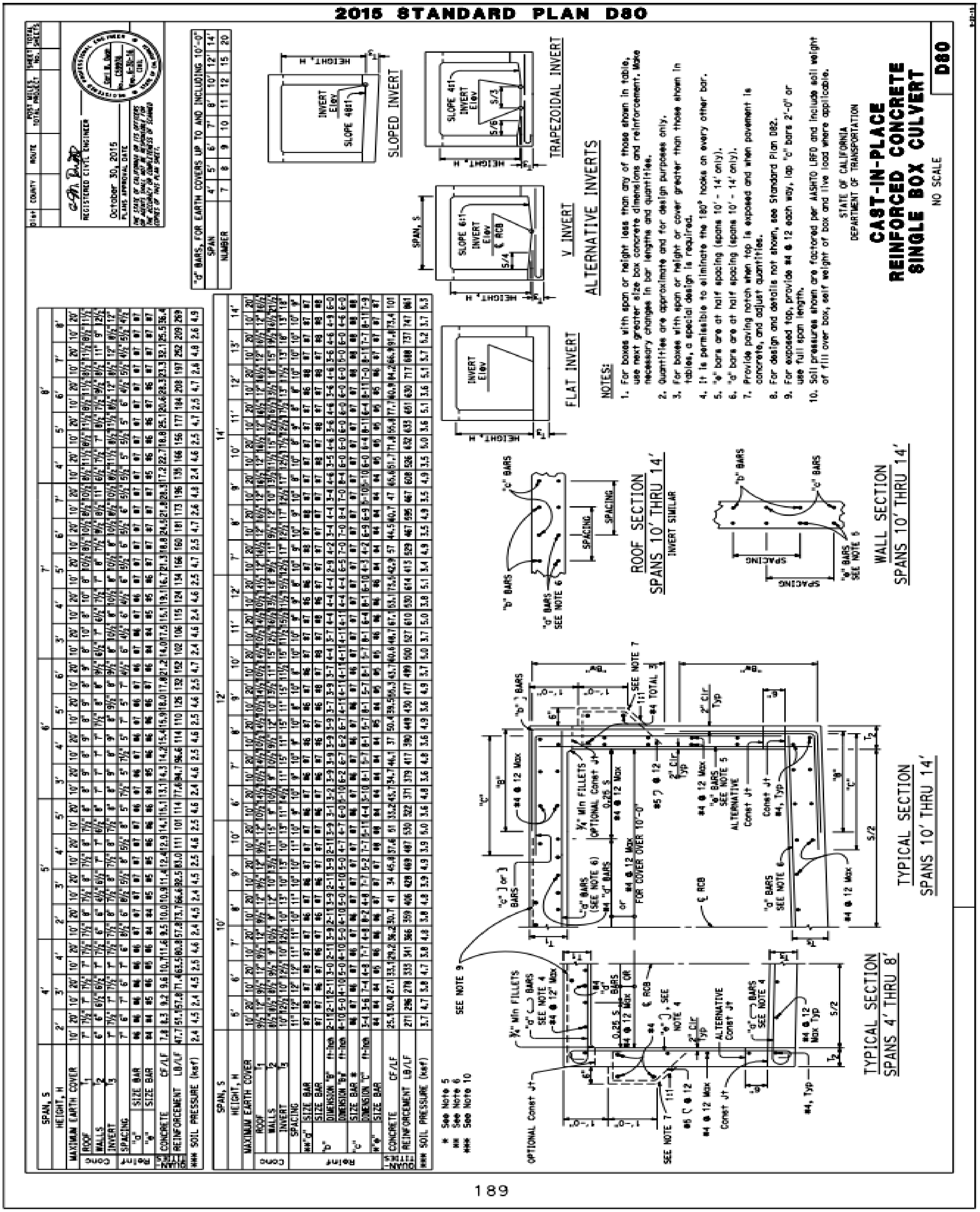



**Figure 10-11: Caltrans Single Cast in Place Reinforced Box (Caltrans, 2015)**

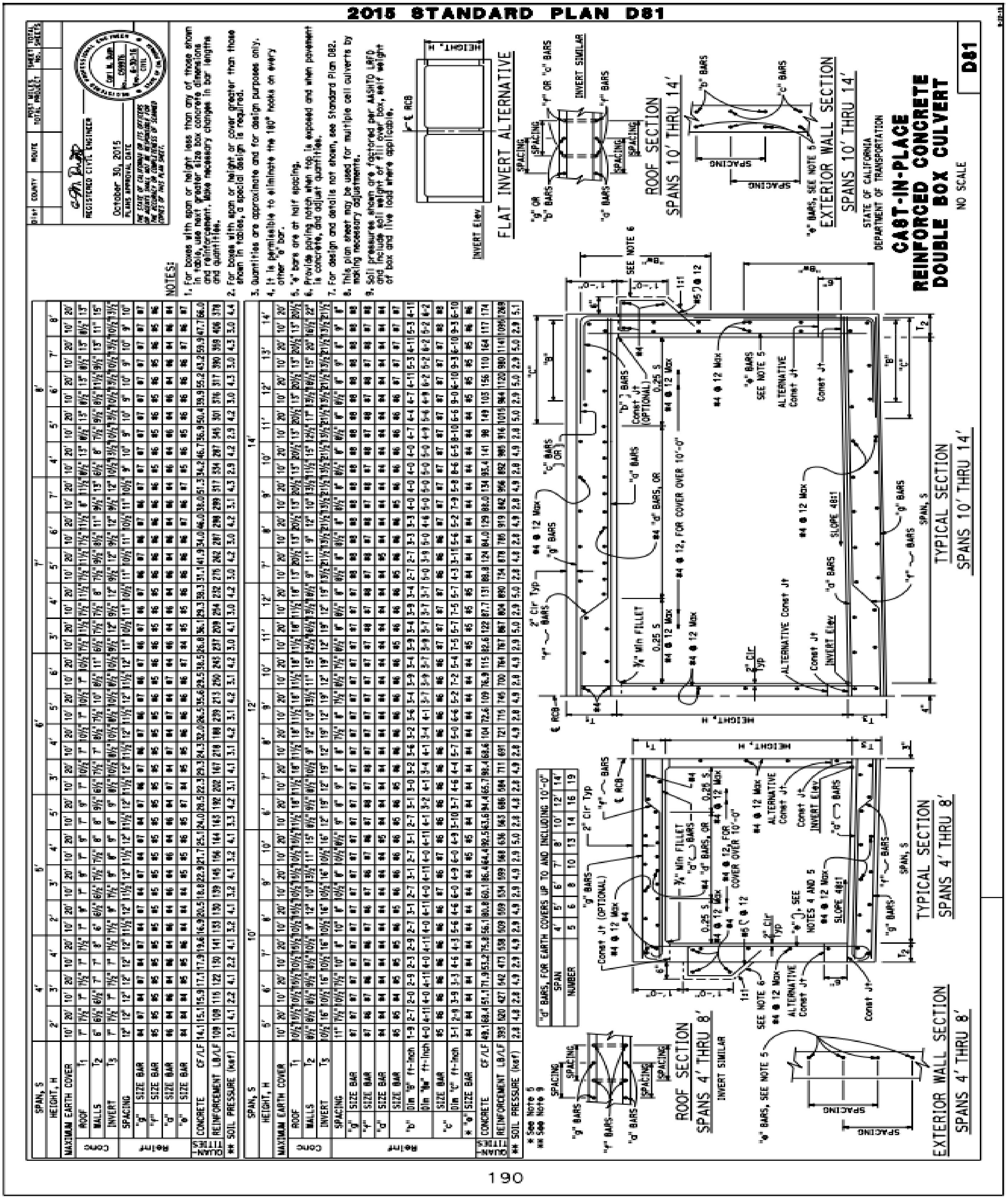



**Figure 10-12: Caltrans Double Cast in Place Reinforced Box (Caltrans, 2015)**

## 10.5 References

American Association of State Highway and Transportation Officials. (2012). *AASHTO LRFD Bridge Design Specifications, Customary U.S. Units (6th Edition) with 2012 and 2013 Interim Revisions; and 2012 Errata*. http://app.knovel.com/hotlink/toc/id:kpAASHTO32/aashto-lrfd-bridge-design/aashto-lrfd-bridge-design

American Concrete Pipe Association. (2011). *Concrete Pipe Design Manual*.

California Department of Transportation. (2004). Bridge Design Specifications.

California Department of Transportation. (2019). *California Amendments to the AASHTO LRFD Bridge Design Specifications (2017 Eighth Edition)*.

California Department of Transportation. (2015). *Standard Plans*.

County of Orange Public Facilities and Resources Department. (2000). *Orange County Flood Control District Design Manual*. https://www.ocflood.com/civicax/filebank/blobdload.aspx?BlobID=26296

# Chapter 11 Floodplains

## 11.1 Introduction

In 1968, Congress created the National Flood Insurance Program (NFIP) to provide flood insurance protection to property owners, in return for local government commitment to sound floodplain management and related flood disaster mitigation efforts. An NFIP flood insurance policy can be obtained directly from the federal government or through an insurance company.

The NFIP was developed by the federal government to offer inexpensive flood insurance. Over 19,000 communities around the U.S. have registered with Federal Emergency Management Agency (FEMA), entitling its members to purchase flood insurance through NFIP authorized insurance agencies.

Orange County and all Cities within the County participate in the NFIP, which provides flood insurance to the County's citizens, flood mitigation assistance, and emergency assistance to flood victims.

Participation in the NFIP is based on an agreement between Orange County and the Federal Government that states that the County will adopt and enforce floodplain management ordinances for new construction to reduce future flood risks to structures in Special Flood Hazard Areas (SFHAs). The agreement provides that the Federal Government will make flood insurance available within Orange County as financial protection against flood losses. Each city within the county should have its own ordinance(s) that regulates building within FEMA Floodplains. In unincorporated Orange County, construction within SFHAs is regulated by Orange County Code of Ordinances Section 7-9-113.

As a member in the FEMA Community Rating System (CRS) program, OCFCD performs various flood mitigation and outreach activities to exceed the minimum requirements of the NFIP as a condition of membership in good standing. Successful participation in CRS has resulted in greater discounts for property owners for flood insurance premiums.

## 11.2 FEMA Special Flood Hazard Areas

FEMA defines Special Flood Hazard Areas (SFHAs) as a high-risk area defined as any land that would be inundated by a flood having a 1-percent chance of occurring in a given year (sometimes referred to as the base flood or 100-year flood). SFHA's are shown on FEMA Flood Insurance Rate Maps (FIRMs). FIRMs are available to download from the FEMA website: https://msc.fema.gov/portal. Property owners within Orange County are required to determine whether their property is located in a FEMA SFHA before commencing any building or land disturbance activity.

## 11.3 Flood Hazard Zones

FIRMs are divided into zones that depict the level of risk from flooding hazards. The technical backup for the FIRM is included in the Flood Insurance Study (FIS) for Orange County, which is available from the FEMA website. Back up data such as hydraulic models are also available from FEMA by filling out a FIS Data Request. The form is available on the FEMA website and can be submitted to FEMA with payment. It is important to note that the information in the FIS is limited to some channels and creeks within Orange County. In cases where detailed data is not available from FEMA, the property owner should contact the City or County to inquire about other available data.

The zones included on FIRMs are as follows:

### 11.3.1 <u>100-year Floodplain</u>

The 100-year floodplain is defined as the floodplain resulting from the runoff from the 100-year storm. FEMA sets its jurisdictional limits to the 100-year event, which is cited as the base flood elevation. The 100-year event is an event that has a 1% chance of occurring in any given year. Jurisdictional limits are defined by horizontal flooding limits using the base flood elevation. The 100-year floodplain is divided by FEMA into the following hazard zones for flood insurance rating purposes:

1. Zone A: No base flood elevations (BFE) determined.
2. Zone AE: Base flood elevations determined.
3. Zone AH: Flood depths of 1 to 3 feet (usually areas of ponding). Base flood elevations determined.
4. Zone AO: Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain). Average depths determined (and velocities determined for alluvial fan floodplains).
5. Zones AE: Areas subject to inundation by the 1-percent-annual-chance flood event determined by detailed methods. BFEs are shown within these zones. (Zone AE is used on new and revised maps in place of Zones A1–A30.)
6. Zone A99: Areas subject to inundation by the 1-percent-annual-chance flood event, but which will ultimately be protected upon completion of an under-construction Federal flood protection system. These are areas of special flood hazard where enough progress has been made on the construction of a protection system, such as dikes, dams, and levees, to consider it complete for insurance rating purposes. Zone A99 may be used only when the flood protection system has reached specified statutory progress toward completion. No BFEs or flood depths are shown.
7. Zone AR: Area that results from the decertification of a previously accredited flood protection system that is determined to be in the process of being restored to provide base flood protection.
8. Zones AR/AE, AR/AH, AR/AO, AR/A1-A30, and AR/A: Dual flood zones that, because of flooding from other water sources that the flood protection system does not contain, will continue to be subject to flooding after the flood protection system is adequately restored.
9. Zones VE, VO: Primary frontal dunes and areas along coasts subject to inundation by the 1-percent-annual-chance flood event with additional hazards due to storm-induced velocity wave action. BFEs derived from detailed hydraulic coastal analyses are shown within these zones. Zone VE is used on new and revised maps in place of Zones V1–V30.
10. Zone X (shaded): Areas of 500-year flood; areas of 100-year flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 100-year flood.
11. Zone X (unshaded): Areas determined to be outside 500-year floodplain.

It is important to note that while the FEMA 100-year floodplain is the regulated floodplain for insurance purposes, flowrates presented in the FEMA analyses do not necessarily correspond to the Orange County 100-year flowrate used for design of OCFCD facilities. Before undertaking the design of flood control facilities, the owner should contact OCPW to determine the correct 100-year flowrate for design.

### 11.3.2 <u>Floodway</u>

- A "Regulatory Floodway" means the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height. Communities must regulate development in these floodways to ensure that there are no increases in upstream flood elevations. For streams and other watercourses where FEMA has provided Base Flood

Elevations (BFEs), but if no floodway has been designated, the community must review floodplain development on a case-by-case basis.
- Floodways are a regulatory tool used by the local municipality to determine areas between the 100-year floodplain and the floodway boundary (referred to as the floodplain fringe), where placement of fill does not increase the base flood elevation by more than 1 foot. Allowable uses in floodways are described in Orange County Code of Ordinances Section 7-9-113.5.
- Because floodways can be dangerous due to swift flowing floodwaters, development is usually not allowed within a floodway, unless an engineer certifies a “no rise” in flood elevations, floodway elevations, or floodway widths.
- The Pad of any habitable structure and other Flood non-flood proofed structures shall be protected from the 100-year storm.
- The design engineer must evaluate the hydraulic impact of the proposed development and prepare a signed, stamped and dated Engineering “No Rise” Certification.

### 11.3.3 Orange County Floodplain Districts

Orange County Code of Ordinances Section 7-9-113 uses floodplain districts to represent the FEMA flood zones. These Floodplain District (“FP District”) regulations apply per section 7-9-48 of the Zoning Code, special flood hazards, those areas of the County which, under present conditions, are subject to periodic flooding and accompanying hazards.

1. The FP-1 is intended to be applied to areas shown as “floodway” on the December 3, 2009 or most current FEMA FIRMs and areas in which the County has determined that a floodway exists.
2. The FP-2 is intended to be applied to areas shown as “A,” “AO,” “AE,” “AH,” and “A99” on the December 3, 2009 FIRM or most current FIRM and areas in which the County has determined a special flood hazard area.
3. The FP-3 is intended to be applied to areas shown as “V”, and “VE” on the December 3, 2009 or most current FEMA FIRMs and areas in which the County has determined a coastal high hazard area.

## 11.4 Construction in Special Flood Hazard Areas

Development may take place within an SFHA, provided the development complies with local floodplain management ordinances, which must meet FEMA requirements. The local requirements are set forth in the OC Code of Ordinances Section 7-9-113. Per the ordinance: “No structure or land shall hereafter be constructed, located, extended, converted, or altered without full compliance with the terms of sections 7-9-113—7-9-113.12 and other applicable regulations. Violation of the requirements (including violations of conditions and safeguards) shall constitute a misdemeanor as governed by section 7-9-154. Nothing herein shall prevent the County of Orange from taking such lawful action as is necessary to prevent or remedy any violation.”

The ordinance also requires:

> “Until a regulatory floodway is adopted, no new construction, substantial improvement, or other development (including fill) shall be permitted within Zones A1-30 and AE, unless it is demonstrated that the cumulative effect of the proposed development, when combined with all other development, will not increase the water surface elevation of the base flood more than one (1) foot.”

In section 7-9-113.9 (e) the ordinance requires:

*"Prior to the issuance of final certificates of use and occupancy for any building, the applicant shall submit to the Manager, Building Inspection the county of Orange "Elevation Certificate" identifying the base flood elevation and certifying that the constructed elevation of the lowest floor, including basements, is at least one foot above the base flood elevation."*

### 11.4.1 Elevation Certificate

The Elevation Certificate is an important administrative tool of the NFIP. It is used to provide elevation information necessary to ensure compliance with community floodplain management ordinances, to determine the proper insurance premium rate, and to support a request for a Letter of Map Amendment (LOMA) or Letter of Map Revision based on fill (LOMR-F) for new and substantially improved structures in a SFHA.

## 11.5 Modification of Flood Zones/Application Process

If a property is determined to be located within a FEMA SFHA after reviewing the appropriate FIRM, there are several approval options available that, if desired and applicable, the landowner must process through FEMA. The landowner must select the permit option that best fits the need of the property and satisfies FEMA requirements. Each permit option requires completion of specific application forms and may require that a registered land surveyor or professional engineer complete the forms. Each permit/application form is identified below by name followed by a brief description of the approval response to be expected from FEMA.

1. **Conditional Letter of Map Amendment (CLOMA)** – A letter from FEMA stating that a proposed structure that is not to be elevated by fill would not be inundated by the 100-year flood if built to the proposed finished floor elevation.
2. **Letter of Map Amendment (LOMA)** – A letter from FEMA stating that an existing structure or parcel of land that has not been elevated by fill would not be inundated by the 100-year flood.
3. **Conditional Letter of Map Revision Based on Fill (CLOMR-F)** – A letter from FEMA stating that a parcel of land or proposed structure that is to be elevated by fill would not be inundated by the 100-year flood if fill is placed on the parcel as proposed and the structure is built as proposed.
4. **Letter of Map Revision Based on Fill (LOMR-F)** – A letter from FEMA stating that an existing structure or parcel of land that has been elevated by fill would not be inundated by the 100-year flood.

Application forms for the four items listed above can be obtained from FEMA by referencing MT-1 FEMA FORM 81-87 SERIES. FEMA's contact address is provided at the end of this section.

1. **Conditional Letter of Map Revision (CLOMR)** – A letter from FEMA commenting on whether a proposed project, if built as proposed, would justify a map revision.
2. **Letter of Map Revision (LOMR)** – A letter from FEMA officially revising the current FIRM to show changes to floodplains, floodways, or flood elevation. Physical changes include watershed development, flood control structures, etc.
3. **Physical Map Revision (PMR**) – A reprinted FIRM incorporating changes to floodplains, floodways, or flood elevations. Because of the time and cost involved to change, reprint, and redistribute a FIRM, a PMR is usually processed when a revision reflects increased flood hazards or large-scope changes.

Application forms for the three items listed above can be obtained from FEMA by referencing MT-2 FEMA FORM 81-89 SERIES. FEMA's contact address is provided at the end of this section.

Projects that successfully receive a conditional letter must apply for a Letter of Map Revision upon completion of construction. The conditional letter allows financing and local approvals. To initiate FEMA review for a specific proposed activity at a specific location, a letter to FEMA requesting one of the “conditional” letters is sent to FEMA along with supporting data, which includes a signed letter from Orange County indicating its concurrence with the request. Supporting data may be in the form of improved methodology, conveyance changes or improved survey data. Improved methodology may be a different technique (model) or adjustments to models used in the effective Flood Insurance Study. Conveyance changes may include proposed conveyance improvements, such as channel lining, shifting, and bridge construction. Improved survey data can include revised data, including placement of fill, as well as new data. Any shift in the FEMA-designated floodway boundaries would be considered as a floodway revision, regardless of whether the shift results in a change that is measurable at the scale of a FIRM panel.

## 11.6 Floodplain Analysis

Required floodplain analysis is generally discussed in the Code of Ordinances Section 7-9-113.9 – Site Development Procedures. The physical alterations of a floodplain or floodway that result in base flood elevation changes require technical and scientific data to change the FIRM as part of the application of a Conditional Letter of Map Revision (CLOMR) and/or Letter of Map Revision (LOMR). Per the County requirements, the application must include a detailed drainage study and plans indicating how site grading, in conjunction with drainage conveyance systems, will provide structures that are safe from flood flows up to and including the base flood.

Floodplain and Floodway analysis can include hydrologic or hydraulic revisions. Requirements for FEMA analyses are described in the MT-2 Forms Instructions (current version) which can be found on the FEMA website. Key components of a floodplain revision application package include: a report describing project and analyses performed (narrative), MT-2 forms, hydrologic analysis (if required), duplicate effective hydraulic model, revised or post project conditions model, proposed (or as-built) project plans, certified topographic floodplain work maps, proposed annotated revised FEMA FIRM maps, digital data of all files, property owner notifications, and public notification. FEMA also charges a fee for reviewing the application package. Current fee structure is listed on the FEMA website.

## 11.7 Acceptable Software

FEMA maintains an extensive list of acceptable software for floodplain revisions. In Orange County, the most common models used for floodplain revisions are as follows:

1. Hydrology: HEC-HMS or other Local Acceptable Models such as AES.
2. Hydraulics: HEC-RAS or WSPGW

Floodplain mapping should be completed using one or more of the following:

1. AutoCAD
2. GIS

A list of current software accepted by FEMA is available on their website. The website should be referenced before selecting the hydrology or hydraulic models for use on a project that impacts a floodplain.

## 11.8 References

Federal Emergency Management Agency. (2011). *Answers to Questions about the NFIP*. https://www.fema.gov/media-library-data/20130726-1438-20490-1905/f084_atq_11aug11.pdf

County of Orange. (1973). *The Codified Ordinances of the County of Orange (Sec. 7-9-113. – FP "Floodplain" District Regulations.).* Order of the Board of Supervisors. https://library.municode.com/ca/orange_county/codes/code_of_ordinances?nodeId=TIT7LAUSBURE_DIV9PL_ART2THCOZOCO_S7-9-113FPFLDIRE

# Appendix A Street Flow Tables

## A.1 Standard Curb Type

## Street Flow Table

Street Half Width = 14'
Curb Type = A2-6"

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.3 | 4.1 | 0 | 3.933 | 3.8 |
| **0.21** | 0.3 | 4.7 | 0 | 3.44 | 4.6 |
| **0.22** | 0.3 | 5.3 | 0 | 2.98 | 5.5 |
| **0.23** | 0.4 | 5.8 | 0 | 2.571 | 6.5 |
| **0.24** | 0.5 | 6.4 | 0 | 2.215 | 7.8 |
| **0.25** | 0.5 | 7 | 0 | 1.912 | 9.2 |
| **0.26** | 0.6 | 7.6 | 0 | 1.654 | 10.8 |
| **0.27** | 0.7 | 8.2 | 0 | 1.436 | 12.7 |
| **0.28** | 0.8 | 8.8 | 0 | 1.251 | 14.7 |
| **0.29** | 0.9 | 9.4 | 0 | 1.094 | 17 |
| **0.3** | 1 | 10 | 0 | 0.961 | 19.5 |
| **0.31** | 1.1 | 10.6 | 0 | 0.0847 | 22.3 |
| **0.32** | 1.2 | 11.2 | 0 | 0.75 | 25.3 |
| **0.33** | 1.3 | 11.8 | 0 | 0.666 | 28.6 |
| **0.34** | 1.4 | 12.3 | 0 | 0.0594 | 32.1 |
| **0.35** | 1.5 | 12.9 | 0 | 0.532 | 36 |
| **0.36** | 1.7 | 13.5 | 0 | 0.477 | 40.1 |
| **EXCEEDS CROWN** | | | | | |
| **0.37** | 3.6 | 27.9 | 0 | 0.424 | 89.6 |
| **0.38** | 3.9 | 27.9 | 0 | 0.364 | 101.5 |
| **0.39** | 4.2 | 27.9 | 0 | 0.315 | 113.9 |
| **0.4** | 4.4 | 28 | 0 | 0.275 | 126.9 |
| **0.41** | 4.7 | 28 | 0 | 0.242 | 140.4 |
| **0.42** | 5 | 28 | 0 | 0.213 | 154.5 |
| **0.43** | 5.3 | 28 | 0 | 0.19 | 169.1 |
| **0.44** | 5.6 | 28 | 0 | 0.169 | 184.2 |
| **0.45** | 5.8 | 28 | 0 | 0.152 | 199.8 |
| **0.46** | 6.1 | 28 | 0 | 0.136 | 216 |
| **0.47** | 6.4 | 28 | 0 | 0.123 | 232.6 |
| **0.48** | 6.7 | 28 | 0 | 0.112 | 249.7 |
| **0.49** | 7 | 28 | 0 | 0.102 | 267.2 |
| **0.5** | 7.2 | 28 | 0 | 0.093 | 285.3 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 7.5 | 28 | 0.5 | 0.088 | 297.7 |
| **0.52** | 7.8 | 28 | 1 | 0.084 | 310.7 |
| **0.53** | 8.1 | 28 | 1.4 | 0.08 | 324.4 |
| **0.54** | 8.4 | 28 | 1.9 | 0.076 | 338.7 |
| **0.55** | 8.8 | 28 | 2.4 | 0.073 | 353.7 |
| **0.56** | 9.1 | 28 | 2.9 | 0.069 | 369.4 |
| **0.57** | 9.4 | 28 | 3.4 | 0.066 | 385.8 |
| **0.58** | 9.8 | 28 | 3.8 | 0.063 | 402.9 |
| **0.59** | 10.1 | 28 | 4.3 | 0.06 | 420.7 |
| **0.6** | 10.5 | 28 | 4.8 | 0.057 | 439.2 |
| **0.61** | 10.9 | 28 | 5.3 | 0.055 | 458.4 |
| **0.62** | 11.3 | 28 | 5.8 | 0.052 | 478.4 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Table

Street Half Width = 15’
Curb Type = A2-6”

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.3 | 4.1 | 0 | 3.933 | 3.8 |
| **0.21** | 0.3 | 4.7 | 0 | 3.44 | 4.6 |
| **0.22** | 0.3 | 5.3 | 0 | 2.98 | 5.5 |
| **0.23** | 0.4 | 5.8 | 0 | 2.571 | 6.5 |
| **0.24** | 0.5 | 6.4 | 0 | 2.215 | 7.8 |
| **0.25** | 0.5 | 7 | 0 | 1.912 | 9.2 |
| **0.26** | 0.6 | 7.6 | 0 | 1.654 | 10.8 |
| **0.27** | 0.7 | 8.2 | 0 | 1.436 | 12.7 |
| **0.28** | 0.8 | 8.8 | 0 | 1.251 | 14.7 |
| **0.29** | 0.9 | 9.4 | 0 | 1.094 | 17 |
| **0.3** | 1 | 10 | 0 | 0.961 | 19.5 |
| **0.31** | 1.1 | 10.6 | 0 | 0.847 | 22.3 |
| **0.32** | 1.2 | 11.2 | 0 | 0.75 | 25.3 |
| **0.33** | 1.3 | 11.8 | 0 | 0.666 | 28.6 |
| **0.34** | 1.4 | 12.3 | 0 | 0.594 | 32.1 |
| **0.35** | 1.5 | 12.9 | 0 | 0.532 | 36 |
| **0.36** | 1.7 | 13.5 | 0 | 0.477 | 40.1 |
| **0.37** | 1.8 | 14.1 | 0 | 0.43 | 44.5 |
| **0.38** | 1.9 | 14.7 | 0 | 0.388 | 49.3 |
| **EXCEEDS CROWN** | | | | | |
| **0.39** | 4.2 | 29.9 | 0 | 0.342 | 110.1 |
| **0.4** | 4.5 | 30 | 0 | 0.297 | 123.5 |
| **0.41** | 4.8 | 30 | 0 | 0.259 | 137.5 |
| **0.42** | 5.1 | 30 | 0 | 0.228 | 152.1 |
| **0.43** | 5.4 | 30 | 0 | 0.202 | 167.2 |
| **0.44** | 5.7 | 30 | 0 | 0.179 | 182.9 |
| **0.45** | 6 | 30 | 0 | 0.16 | 199.2 |
| **0.46** | 6.3 | 30 | 0 | 0.144 | 216 |
| **0.47** | 6.6 | 30 | 0 | 0.13 | 233.4 |
| **0.48** | 6.9 | 30 | 0 | 0.117 | 251.3 |
| **0.49** | 7.2 | 30 | 0 | 0.106 | 269.7 |
| **0.5** | 7.5 | 30 | 0 | 0.097 | 288.6 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 7.8 | 30 | 0.5 | 0.092 | 302.2 |
| **0.52** | 8.1 | 30 | 1 | 0.087 | 316.4 |
| **0.53** | 8.4 | 30 | 1.4 | 0.083 | 331.4 |
| **0.54** | 8.8 | 30 | 1.9 | 0.079 | 347 |
| **0.55** | 9.1 | 30 | 2.4 | 0.075 | 363.2 |
| **0.56** | 9.5 | 30 | 2.9 | 0.071 | 380.2 |
| **0.57** | 9.8 | 30 | 3.4 | 0.067 | 397.9 |
| **0.58** | 10.2 | 30 | 3.8 | 0.064 | 416.2 |
| **0.59** | 10.6 | 30 | 4.3 | 0.061 | 435.3 |
| **0.6** | 11 | 30 | 4.8 | 0.058 | 455.1 |
| **0.61** | 11.4 | 30 | 5.3 | 0.055 | 475.7 |
| **0.62** | 11.8 | 30 | 5.8 | 0.053 | 497 |
| **0.63** | 12.2 | 30 | 6.2 | 0.05 | 519.1 |
| **0.64** | 12.6 | 30 | 6.7 | 0.048 | 542 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Table

Street Half Width = 16'
Curb Type = A2-6"

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.3 | 4.1 | 0 | 3.933 | 3.8 |
| **0.21** | 0.3 | 4.7 | 0 | 3.44 | 4.6 |
| **0.22** | 0.3 | 5.3 | 0 | 2.98 | 5.5 |
| **0.23** | 0.4 | 5.8 | 0 | 2.571 | 6.5 |
| **0.24** | 0.5 | 6.4 | 0 | 2.215 | 7.8 |
| **0.25** | 0.5 | 7 | 0 | 1.912 | 9.2 |
| **0.26** | 0.6 | 7.6 | 0 | 1.654 | 10.8 |
| **0.27** | 0.7 | 8.2 | 0 | 1.436 | 12.7 |
| **0.28** | 0.8 | 8.8 | 0 | 1.251 | 14.7 |
| **0.29** | 0.9 | 9.4 | 0 | 1.094 | 17 |
| **0.3** | 1 | 10 | 0 | 0.961 | 19.5 |
| **0.31** | 1.1 | 10.6 | 0 | 0.847 | 22.3 |
| **0.32** | 1.2 | 11.2 | 0 | 0.75 | 25.3 |
| **0.33** | 1.3 | 11.8 | 0 | 0.666 | 28.6 |
| **0.34** | 1.4 | 12.3 | 0 | 0.594 | 32.1 |
| **0.35** | 1.5 | 12.9 | 0 | 0.532 | 36 |
| **0.36** | 1.7 | 13.5 | 0 | 0.477 | 40.1 |
| **0.37** | 1.8 | 14.1 | 0 | 0.43 | 44.5 |
| **0.38** | 1.9 | 14.7 | 0 | 0.388 | 49.3 |
| **0.39** | 2.1 | 15.3 | 0 | 0.351 | 54.3 |
| **0.4** | 2.2 | 15.9 | 0 | 0.319 | 59.7 |
| **EXCEEDS CROWN** | | | | | |
| **0.41** | 4.8 | 32 | 0 | 0.28 | 133.4 |
| **0.42** | 5.1 | 32 | 0 | 0.245 | 148.4 |
| **0.43** | 5.5 | 32 | 0 | 0.216 | 164 |
| **0.44** | 5.8 | 32 | 0 | 0.191 | 180.3 |
| **0.45** | 6.1 | 32 | 0 | 0.17 | 197.1 |
| **0.46** | 6.4 | 32 | 0 | 0.152 | 214.6 |
| **0.47** | 6.7 | 32 | 0 | 0.137 | 232.6 |
| **0.48** | 7.1 | 32 | 0 | 0.123 | 251.2 |
| **0.49** | 7.4 | 32 | 0 | 0.112 | 270.3 |
| **0.5** | 7.7 | 32 | 0 | 0.101 | 290 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 8 | 32 | 0.5 | 0.096 | 304.8 |
| **0.52** | 8.4 | 32 | 1 | 0.091 | 320.2 |
| **0.53** | 8.7 | 32 | 1.4 | 0.086 | 336.3 |
| **0.54** | 9.1 | 32 | 1.9 | 0.081 | 353.1 |
| **0.55** | 9.4 | 32 | 2.4 | 0.077 | 370.6 |
| **0.56** | 9.8 | 32 | 2.9 | 0.073 | 388.8 |
| **0.57** | 10.2 | 32 | 3.4 | 0.069 | 407.7 |
| **0.58** | 10.6 | 32 | 3.8 | 0.065 | 427.3 |
| **0.59** | 11 | 32 | 4.3 | 0.062 | 447.7 |
| **0.6** | 11.4 | 32 | 4.8 | 0.059 | 468.8 |
| **0.61** | 11.8 | 32 | 5.3 | 0.056 | 490.7 |
| **0.62** | 12.2 | 32 | 5.8 | 0.053 | 513.3 |
| **0.63** | 12.7 | 32 | 6.2 | 0.051 | 536.8 |
| **0.64** | 13.1 | 32 | 6.7 | 0.048 | 561 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Table

Street Half Width = 17'
Curb Type = A2-6"

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.3 | 4.1 | 0 | 3.933 | 3.8 |
| **0.21** | 0.3 | 4.7 | 0 | 3.44 | 4.6 |
| **0.22** | 0.3 | 5.3 | 0 | 2.98 | 5.5 |
| **0.23** | 0.4 | 5.8 | 0 | 2.571 | 6.5 |
| **0.24** | 0.5 | 6.4 | 0 | 2.215 | 7.8 |
| **0.25** | 0.5 | 7 | 0 | 1.912 | 9.2 |
| **0.26** | 0.6 | 7.6 | 0 | 1.654 | 10.8 |
| **0.27** | 0.7 | 8.2 | 0 | 1.436 | 12.7 |
| **0.28** | 0.8 | 8.8 | 0 | 1.251 | 14.7 |
| **0.29** | 0.9 | 9.4 | 0 | 1.094 | 17 |
| **0.3** | 1 | 10 | 0 | 0.961 | 19.5 |
| **0.31** | 1.1 | 10.6 | 0 | 0.847 | 22.3 |
| **0.32** | 1.2 | 11.2 | 0 | 0.75 | 25.3 |
| **0.33** | 1.3 | 11.8 | 0 | 0.666 | 28.6 |
| **0.34** | 1.4 | 12.3 | 0 | 0.594 | 32.1 |
| **0.35** | 1.5 | 12.9 | 0 | 0.532 | 36 |
| **0.36** | 1.7 | 13.5 | 0 | 0.477 | 40.1 |
| **0.37** | 1.8 | 14.1 | 0 | 0.43 | 44.5 |
| **0.38** | 1.9 | 14.7 | 0 | 0.388 | 49.3 |
| **0.39** | 2.1 | 15.3 | 0 | 0.351 | 54.3 |
| **0.4** | 2.2 | 15.9 | 0 | 0.319 | 59.7 |
| **0.41** | 2.4 | 16.5 | 0 | 0.291 | 65.5 |
| **EXCEEDS CROWN** | | | | | |
| **0.42** | 5.2 | 34 | 0 | 0.264 | 143.5 |
| **0.43** | 5.5 | 34 | 0 | 0.231 | 159.6 |
| **0.44** | 5.8 | 34 | 0 | 0.204 | 176.3 |
| **0.45** | 6.2 | 34 | 0 | 0.181 | 193.6 |
| **0.46** | 6.5 | 34 | 0 | 0.161 | 211.6 |
| **0.47** | 6.9 | 34 | 0 | 0.145 | 230.2 |
| **0.48** | 7.2 | 34 | 0 | 0.13 | 249.5 |
| **0.49** | 7.5 | 34 | 0 | 0.117 | 269.3 |
| **0.5** | 7.9 | 34 | 0 | 0.106 | 289.8 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 8.2 | 34 | 0.5 | 0.1 | 305.6 |
| **0.52** | 8.6 | 34 | 1 | 0.094 | 322.2 |
| **0.53** | 8.9 | 34 | 1.4 | 0.089 | 339.4 |
| **0.54** | 9.3 | 34 | 1.9 | 0.084 | 357.3 |
| **0.55** | 9.7 | 34 | 2.4 | 0.079 | 375.9 |
| **0.56** | 10.1 | 34 | 2.9 | 0.075 | 395.3 |
| **0.57** | 10.5 | 34 | 3.4 | 0.071 | 415.4 |
| **0.58** | 10.9 | 34 | 3.8 | 0.067 | 436.2 |
| **0.59** | 11.3 | 34 | 4.3 | 0.063 | 457.8 |
| **0.6** | 11.8 | 34 | 4.8 | 0.06 | 480.2 |
| **0.61** | 12.2 | 34 | 5.3 | 0.057 | 503.4 |
| **0.62** | 12.6 | 34 | 5.8 | 0.054 | 527.3 |
| **0.63** | 13.1 | 34 | 6.2 | 0.051 | 552.1 |
| **0.64** | 13.6 | 34 | 6.7 | 0.049 | 577.6 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Table

Street Half Width = 18'
Curb Type = A2-6"

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.3 | 4.1 | 0 | 3.933 | 3.8 |
| **0.21** | 0.3 | 4.7 | 0 | 3.44 | 4.6 |
| **0.22** | 0.3 | 5.3 | 0 | 2.98 | 5.5 |
| **0.23** | 0.4 | 5.8 | 0 | 2.571 | 6.5 |
| **0.24** | 0.5 | 6.4 | 0 | 2.215 | 7.8 |
| **0.25** | 0.5 | 7 | 0 | 1.912 | 9.2 |
| **0.26** | 0.6 | 7.6 | 0 | 1.654 | 10.8 |
| **0.27** | 0.7 | 8.2 | 0 | 1.436 | 12.7 |
| **0.28** | 0.8 | 8.8 | 0 | 1.251 | 14.7 |
| **0.29** | 0.9 | 9.4 | 0 | 1.094 | 17 |
| **0.3** | 1 | 10 | 0 | 0.961 | 19.5 |
| **0.31** | 1.1 | 10.6 | 0 | 0.847 | 22.3 |
| **0.32** | 1.2 | 11.2 | 0 | 0.75 | 25.3 |
| **0.33** | 1.3 | 11.8 | 0 | 0.666 | 28.6 |
| **0.34** | 1.4 | 12.3 | 0 | 0.594 | 32.1 |
| **0.35** | 1.5 | 12.9 | 0 | 0.532 | 36 |
| **0.36** | 1.7 | 13.5 | 0 | 0.477 | 40.1 |
| **0.37** | 1.8 | 14.1 | 0 | 0.43 | 44.5 |
| **0.38** | 1.9 | 14.7 | 0 | 0.388 | 49.3 |
| **0.39** | 2.1 | 15.3 | 0 | 0.351 | 54.3 |
| **0.4** | 2.2 | 15.9 | 0 | 0.319 | 59.7 |
| **0.41** | 2.4 | 16.5 | 0 | 0.291 | 65.5 |
| **0.42** | 2.6 | 17.1 | 0 | 0.265 | 71.5 |
| **0.43** | 2.8 | 17.7 | 0 | 0.243 | 78 |
| **EXCEEDS CROWN** | | | | | |
| **0.44** | 5.9 | 36 | 0 | 0.218 | 171.1 |
| **0.45** | 6.2 | 36 | 0 | 0.193 | 188.9 |
| **0.46** | 6.6 | 36 | 0 | 0.172 | 207.3 |
| **0.47** | 6.9 | 36 | 0 | 0.153 | 226.5 |
| **0.48** | 7.3 | 36 | 0 | 0.137 | 246.3 |
| **0.49** | 7.7 | 36 | 0 | 0.124 | 266.7 |
| **0.5** | 8 | 36 | 0 | 0.112 | 287.8 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 8.4 | 36 | 0.5 | 0.105 | 304.7 |
| **0.52** | 8.8 | 36 | 1 | 0.098 | 322.3 |
| **0.53** | 9.1 | 36 | 1.4 | 0.092 | 340.6 |
| **0.54** | 9.5 | 36 | 1.9 | 0.087 | 359.6 |
| **0.55** | 9.9 | 36 | 2.4 | 0.082 | 379.3 |
| **0.56** | 10.4 | 36 | 2.9 | 0.077 | 399.8 |
| **0.57** | 10.8 | 36 | 3.4 | 0.073 | 421 |
| **0.58** | 11.2 | 36 | 3.8 | 0.069 | 443 |
| **0.59** | 11.7 | 36 | 4.3 | 0.065 | 465.8 |
| **0.6** | 12.1 | 36 | 4.8 | 0.061 | 489.4 |
| **0.61** | 12.6 | 36 | 5.3 | 0.058 | 513.8 |
| **0.62** | 13 | 36 | 5.8 | 0.055 | 539 |
| **0.63** | 13.5 | 36 | 6.2 | 0.052 | 565 |
| **0.64** | 14 | 36 | 6.7 | 0.049 | 591.9 |
| **0.65** | 14.5 | 36 | 7.2 | 0.047 | 619.6 |
| **0.66** | 15 | 36 | 7.7 | 0.044 | 648.2 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Table

Street Half Width = 19’
Curb Type = A2-6”

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.3 | 4.1 | 0 | 3.933 | 3.8 |
| **0.21** | 0.3 | 4.7 | 0 | 3.44 | 4.6 |
| **0.22** | 0.3 | 5.3 | 0 | 2.98 | 5.5 |
| **0.23** | 0.4 | 5.8 | 0 | 2.571 | 6.5 |
| **0.24** | 0.5 | 6.4 | 0 | 2.215 | 7.8 |
| **0.25** | 0.5 | 7 | 0 | 1.912 | 9.2 |
| **0.26** | 0.6 | 7.6 | 0 | 1.654 | 10.8 |
| **0.27** | 0.7 | 8.2 | 0 | 1.436 | 12.7 |
| **0.28** | 0.8 | 8.8 | 0 | 1.251 | 14.7 |
| **0.29** | 0.9 | 9.4 | 0 | 1.094 | 17 |
| **0.3** | 1 | 10 | 0 | 0.961 | 19.5 |
| **0.31** | 1.1 | 10.6 | 0 | 0.847 | 22.3 |
| **0.32** | 1.2 | 11.2 | 0 | 0.75 | 25.3 |
| **0.33** | 1.3 | 11.8 | 0 | 0.666 | 28.6 |
| **0.34** | 1.4 | 12.3 | 0 | 0.594 | 32.1 |
| **0.35** | 1.5 | 12.9 | 0 | 0.532 | 36 |
| **0.36** | 1.7 | 13.5 | 0 | 0.477 | 40.1 |
| **0.37** | 1.8 | 14.1 | 0 | 0.43 | 44.5 |
| **0.38** | 1.9 | 14.7 | 0 | 0.388 | 49.3 |
| **0.39** | 2.1 | 15.3 | 0 | 0.351 | 54.3 |
| **0.4** | 2.2 | 15.9 | 0 | 0.319 | 59.7 |
| **0.41** | 2.4 | 16.5 | 0 | 0.291 | 65.5 |
| **0.42** | 2.6 | 17.1 | 0 | 0.265 | 71.5 |
| **0.43** | 2.8 | 17.7 | 0 | 0.243 | 78 |
| **0.44** | 2.9 | 18.2 | 0 | 0.223 | 84.8 |
| **0.45** | 3.1 | 18.8 | 0 | 0.205 | 91.9 |
| **EXCEEDS CROWN** | | | | | |
| **0.46** | 6.6 | 38 | 0 | 0.183 | 201.7 |
| **0.47** | 7 | 38 | 0 | 0.163 | 221.3 |
| **0.48** | 7.4 | 38 | 0 | 0.146 | 241.7 |
| **0.49** | 7.8 | 38 | 0 | 0.131 | 262.7 |
| **0.5** | 8.1 | 38 | 0 | 0.118 | 284.4 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 8.5 | 38 | 0.5 | 0.11 | 302.2 |
| **0.52** | 8.9 | 38 | 1 | 0.103 | 320.7 |
| **0.53** | 9.3 | 38 | 1.4 | 0.096 | 340 |
| **0.54** | 9.7 | 38 | 1.9 | 0.09 | 360 |
| **0.55** | 10.2 | 38 | 2.4 | 0.085 | 380.8 |
| **0.56** | 10.6 | 38 | 2.9 | 0.079 | 402.3 |
| **0.57** | 11 | 38 | 3.4 | 0.075 | 424.7 |
| **0.58** | 11.5 | 38 | 3.8 | 0.07 | 447.8 |
| **0.59** | 11.9 | 38 | 4.3 | 0.066 | 471.7 |
| **0.6** | 12.4 | 38 | 4.8 | 0.063 | 496.5 |
| **0.61** | 12.9 | 38 | 5.3 | 0.059 | 522 |
| **0.62** | 13.4 | 38 | 5.8 | 0.056 | 548.5 |
| **0.63** | 13.9 | 38 | 6.2 | 0.053 | 575.7 |
| **0.64** | 14.4 | 38 | 6.7 | 0.05 | 603.9 |
| **0.65** | 14.9 | 38 | 7.2 | 0.047 | 632.9 |
| **0.66** | 15.4 | 38 | 7.7 | 0.045 | 662.8 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Table

Street Half Width = 20'
Curb Type = A2-6"

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.3 | 4.1 | 0 | 3.933 | 3.8 |
| **0.21** | 0.3 | 4.7 | 0 | 3.44 | 4.6 |
| **0.22** | 0.3 | 5.3 | 0 | 2.98 | 5.5 |
| **0.23** | 0.4 | 5.8 | 0 | 2.571 | 6.5 |
| **0.24** | 0.5 | 6.4 | 0 | 2.215 | 7.8 |
| **0.25** | 0.5 | 7 | 0 | 1.912 | 9.2 |
| **0.26** | 0.6 | 7.6 | 0 | 1.654 | 10.8 |
| **0.27** | 0.7 | 8.2 | 0 | 1.436 | 12.7 |
| **0.28** | 0.8 | 8.8 | 0 | 1.251 | 14.7 |
| **0.29** | 0.9 | 9.4 | 0 | 1.094 | 17 |
| **0.3** | 1 | 10 | 0 | 0.961 | 19.5 |
| **0.31** | 1.1 | 10.6 | 0 | 0.847 | 22.3 |
| **0.32** | 1.2 | 11.2 | 0 | 0.75 | 25.3 |
| **0.33** | 1.3 | 11.8 | 0 | 0.666 | 28.6 |
| **0.34** | 1.4 | 12.3 | 0 | 0.594 | 32.1 |
| **0.35** | 1.5 | 12.9 | 0 | 0.532 | 36 |
| **0.36** | 1.7 | 13.5 | 0 | 0.477 | 40.1 |
| **0.37** | 1.8 | 14.1 | 0 | 0.43 | 44.5 |
| **0.38** | 1.9 | 14.7 | 0 | 0.388 | 49.3 |
| **0.39** | 2.1 | 15.3 | 0 | 0.351 | 54.3 |
| **0.4** | 2.2 | 15.9 | 0 | 0.319 | 59.7 |
| **0.41** | 2.4 | 16.5 | 0 | 0.291 | 65.5 |
| **0.42** | 2.6 | 17.1 | 0 | 0.265 | 71.5 |
| **0.43** | 2.8 | 17.7 | 0 | 0.243 | 78 |
| **0.44** | 2.9 | 18.2 | 0 | 0.223 | 84.8 |
| **0.45** | 3.1 | 18.8 | 0 | 0.205 | 91.9 |
| **0.46** | 3.3 | 19.4 | 0 | 0.188 | 99.5 |
| **EXCEEDS CROWN** | | | | | |
| **0.47** | 7 | 40 | 0 | 0.173 | 215 |
| **0.48** | 7.4 | 40 | 0 | 0.155 | 235.7 |
| **0.49** | 7.8 | 40 | 0 | 0.138 | 257.2 |
| **0.5** | 8.1 | 40 | 0 | 0.124 | 279.4 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 8.6 | 40 | 0.5 | 0.116 | 298.1 |
| **0.52** | 9 | 40 | 1 | 0.108 | 317.5 |
| **0.53** | 9.5 | 40 | 1.4 | 0.1 | 337.7 |
| **0.54** | 9.9 | 40 | 1.9 | 0.094 | 358.7 |
| **0.55** | 10.3 | 40 | 2.4 | 0.088 | 380.4 |
| **0.56** | 10.8 | 40 | 2.9 | 0.082 | 403 |
| **0.57** | 11.2 | 40 | 3.4 | 0.077 | 426.3 |
| **0.58** | 11.7 | 40 | 3.8 | 0.072 | 450.5 |
| **0.59** | 12.2 | 40 | 4.3 | 0.068 | 475.5 |
| **0.6** | 12.7 | 40 | 4.8 | 0.064 | 501.4 |
| **0.61** | 13.2 | 40 | 5.3 | 0.06 | 528.1 |
| **0.62** | 13.7 | 40 | 5.8 | 0.057 | 555.7 |
| **0.63** | 14.2 | 40 | 6.2 | 0.054 | 584.2 |
| **0.64** | 14.8 | 40 | 6.7 | 0.051 | 613.5 |
| **0.65** | 15.3 | 40 | 7.2 | 0.048 | 643.8 |
| **0.66** | 15.8 | 40 | 7.7 | 0.046 | 675 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Table

Street Half Width = 21’
Curb Type = A2-6”

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.3 | 4.1 | 0 | 3.933 | 3.8 |
| **0.21** | 0.3 | 4.7 | 0 | 3.44 | 4.6 |
| **0.22** | 0.3 | 5.3 | 0 | 2.98 | 5.5 |
| **0.23** | 0.4 | 5.8 | 0 | 2.571 | 6.5 |
| **0.24** | 0.5 | 6.4 | 0 | 2.215 | 7.8 |
| **0.25** | 0.5 | 7 | 0 | 1.912 | 9.2 |
| **0.26** | 0.6 | 7.6 | 0 | 1.654 | 10.8 |
| **0.27** | 0.7 | 8.2 | 0 | 1.436 | 12.7 |
| **0.28** | 0.8 | 8.8 | 0 | 1.251 | 14.7 |
| **0.29** | 0.9 | 9.4 | 0 | 1.094 | 17 |
| **0.3** | 1 | 10 | 0 | 0.961 | 19.5 |
| **0.31** | 1.1 | 10.6 | 0 | 0.847 | 22.3 |
| **0.32** | 1.2 | 11.2 | 0 | 0.75 | 25.3 |
| **0.33** | 1.3 | 11.8 | 0 | 0.666 | 28.6 |
| **0.34** | 1.4 | 12.3 | 0 | 0.594 | 32.1 |
| **0.35** | 1.5 | 12.9 | 0 | 0.532 | 36 |
| **0.36** | 1.7 | 13.5 | 0 | 0.477 | 40.1 |
| **0.37** | 1.8 | 14.1 | 0 | 0.43 | 44.5 |
| **0.38** | 1.9 | 14.7 | 0 | 0.388 | 49.3 |
| **0.39** | 2.1 | 15.3 | 0 | 0.351 | 54.3 |
| **0.4** | 2.2 | 15.9 | 0 | 0.319 | 59.7 |
| **0.41** | 2.4 | 16.5 | 0 | 0.291 | 65.5 |
| **0.42** | 2.6 | 17.1 | 0 | 0.265 | 71.5 |
| **0.43** | 2.8 | 17.7 | 0 | 0.243 | 78 |
| **0.44** | 2.9 | 18.2 | 0 | 0.223 | 84.8 |
| **0.45** | 3.1 | 18.8 | 0 | 0.205 | 91.9 |
| **0.46** | 3.3 | 19.4 | 0 | 0.188 | 99.5 |
| **0.47** | 3.5 | 20 | 0 | 0.174 | 107.4 |
| **0.48** | 3.7 | 20.6 | 0 | 0.161 | 115.7 |
| **EXCEEDS CROWN** | | | | | |
| **0.49** | 7.8 | 42 | 0 | 0.147 | 250.4 |
| **0.5** | 8.3 | 42 | 0 | 0.132 | 273.1 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 8.7 | 42 | 0.5 | 0.122 | 292.5 |
| **0.52** | 9.1 | 42 | 1 | 0.113 | 312.8 |
| **0.53** | 9.6 | 42 | 1.4 | 0.105 | 333.8 |
| **0.54** | 10 | 42 | 1.9 | 0.098 | 355.7 |
| **0.55** | 10.5 | 42 | 2.4 | 0.091 | 378.3 |
| **0.56** | 10.9 | 42 | 2.9 | 0.085 | 401.8 |
| **0.57** | 11.4 | 42 | 3.4 | 0.08 | 426.2 |
| **0.58** | 11.9 | 42 | 3.8 | 0.075 | 451.3 |
| **0.59** | 12.4 | 42 | 4.3 | 0.07 | 477.4 |
| **0.6** | 12.9 | 42 | 4.8 | 0.066 | 504.3 |
| **0.61** | 13.5 | 42 | 5.3 | 0.062 | 532.1 |
| **0.62** | 14 | 42 | 5.8 | 0.058 | 560.8 |
| **0.63** | 14.5 | 42 | 6.2 | 0.055 | 590.5 |
| **0.64** | 15.1 | 42 | 6.7 | 0.052 | 621 |
| **0.65** | 15.6 | 42 | 7.2 | 0.049 | 652.4 |
| **0.66** | 16.2 | 42 | 7.7 | 0.046 | 684.8 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Table

Street Half Width = 22’
Curb Type = A2-6”

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.3 | 4.1 | 0 | 3.933 | 3.8 |
| **0.21** | 0.3 | 4.7 | 0 | 3.44 | 4.6 |
| **0.22** | 0.3 | 5.3 | 0 | 2.98 | 5.5 |
| **0.23** | 0.4 | 5.8 | 0 | 2.571 | 6.5 |
| **0.24** | 0.5 | 6.4 | 0 | 2.215 | 7.8 |
| **0.25** | 0.5 | 7 | 0 | 1.912 | 9.2 |
| **0.26** | 0.6 | 7.6 | 0 | 1.654 | 10.8 |
| **0.27** | 0.7 | 8.2 | 0 | 1.436 | 12.7 |
| **0.28** | 0.8 | 8.8 | 0 | 1.251 | 14.7 |
| **0.29** | 0.9 | 9.4 | 0 | 1.094 | 17 |
| **0.3** | 1 | 10 | 0 | 0.961 | 19.5 |
| **0.31** | 1.1 | 10.6 | 0 | 0.847 | 22.3 |
| **0.32** | 1.2 | 11.2 | 0 | 0.75 | 25.3 |
| **0.33** | 1.3 | 11.8 | 0 | 0.666 | 28.6 |
| **0.34** | 1.4 | 12.3 | 0 | 0.594 | 32.1 |
| **0.35** | 1.5 | 12.9 | 0 | 0.532 | 36 |
| **0.36** | 1.7 | 13.5 | 0 | 0.477 | 40.1 |
| **0.37** | 1.8 | 14.1 | 0 | 0.43 | 44.5 |
| **0.38** | 1.9 | 14.7 | 0 | 0.388 | 49.3 |
| **0.39** | 2.1 | 15.3 | 0 | 0.351 | 54.3 |
| **0.4** | 2.2 | 15.9 | 0 | 0.319 | 59.7 |
| **0.41** | 2.4 | 16.5 | 0 | 0.291 | 65.5 |
| **0.42** | 2.6 | 17.1 | 0 | 0.265 | 71.5 |
| **0.43** | 2.8 | 17.7 | 0 | 0.243 | 78 |
| **0.44** | 2.9 | 18.2 | 0 | 0.223 | 84.8 |
| **0.45** | 3.1 | 18.8 | 0 | 0.205 | 91.9 |
| **0.46** | 3.3 | 19.4 | 0 | 0.188 | 99.5 |
| **0.47** | 3.5 | 20 | 0 | 0.174 | 107.4 |
| **0.48** | 3.7 | 20.6 | 0 | 0.161 | 115.7 |
| **0.49** | 3.9 | 21.2 | 0 | 0.0149 | 124.4 |
| **0.5** | 4.1 | 21.8 | 0 | 0.138 | 133.5 |
| **EXCEEDS CURB AND CROWN** | | | | | |
| **0.51** | 8.7 | 44 | 0.5 | 0.129 | 285.5 |
| **0.52** | 9.2 | 44 | 1 | 0.119 | 306.5 |
| **0.53** | 9.6 | 44 | 1.4 | 0.11 | 328.3 |
| **0.54** | 10.1 | 44 | 1.9 | 0.102 | 351 |
| **0.55** | 10.6 | 44 | 2.4 | 0.095 | 374.5 |
| **0.56** | 11.1 | 44 | 2.9 | 0.089 | 398.9 |
| **0.57** | 11.6 | 44 | 3.4 | 0.083 | 424.1 |
| **0.58** | 12.1 | 44 | 3.8 | 0.077 | 450.3 |
| **0.59** | 12.6 | 44 | 4.3 | 0.072 | 477.3 |
| **0.6** | 13.1 | 44 | 4.8 | 0.068 | 505.2 |
| **0.61** | 13.7 | 44 | 5.3 | 0.064 | 534.1 |
| **0.62** | 14.2 | 44 | 5.8 | 0.06 | 563.8 |
| **0.63** | 14.8 | 44 | 6.2 | 0.056 | 594.5 |
| **0.64** | 15.4 | 44 | 6.7 | 0.053 | 626.2 |
| **0.65** | 15.9 | 44 | 7.2 | 0.05 | 658.8 |
| **0.66** | 16.5 | 44 | 7.7 | 0.047 | 692.4 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Table

Street Half Width = 32’
Curb Type = A2-8”

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 2.1 | 0 | 2.055 | 4.6 |
| **0.21** | 0.2 | 2.7 | 0 | 2.223 | 4.7 |
| **0.22** | 0.3 | 3.3 | 0 | 2.239 | 5 |
| **0.23** | 0.3 | 3.8 | 0 | 2.153 | 5.5 |
| **0.24** | 0.3 | 4.4 | 0 | 2.007 | 6.2 |
| **0.25** | 0.4 | 5 | 0 | 1.835 | 7 |
| **0.26** | 0.5 | 5.6 | 0 | 1.656 | 8.1 |
| **0.27** | 0.5 | 6.2 | 0 | 1.484 | 9.3 |
| **0.28** | 0.6 | 6.8 | 0 | 1.323 | 10.7 |
| **0.29** | 0.6 | 7.4 | 0 | 1.177 | 12.3 |
| **0.3** | 0.7 | 8 | 0 | 1.047 | 14.1 |
| **0.31** | 0.8 | 8.6 | 0 | 0.931 | 16.1 |
| **0.32** | 0.9 | 9.2 | 0 | 0.83 | 18.4 |
| **0.33** | 1 | 9.8 | 0 | 0.74 | 20.9 |
| **0.34** | 1.1 | 10.3 | 0 | 0.662 | 23.6 |
| **0.35** | 1.2 | 10.9 | 0 | 0.594 | 26.6 |
| **0.36** | 1.3 | 11.5 | 0 | 0.533 | 29.8 |
| **0.37** | 1.4 | 12.1 | 0 | 0.48 | 33.4 |
| **0.38** | 1.5 | 12.7 | 0 | 0.434 | 37.2 |
| **0.39** | 1.7 | 13.3 | 0 | 0.393 | 41.2 |
| **0.4** | 1.8 | 13.9 | 0 | 0.356 | 45.6 |
| **0.41** | 2 | 14.5 | 0 | 0.324 | 50.3 |
| **0.42** | 2.1 | 15.1 | 0 | 0.296 | 55.3 |
| **0.43** | 2.3 | 15.7 | 0 | 0.27 | 60.7 |
| **0.44** | 2.4 | 16.3 | 0 | 0.247 | 66.3 |
| **0.45** | 2.6 | 16.8 | 0 | 0.227 | 72.3 |
| **0.46** | 2.8 | 17.4 | 0 | 0.209 | 78.7 |
| **0.47** | 2.9 | 18 | 0 | 0.192 | 85.4 |
| **0.48** | 3.1 | 18.6 | 0 | 0.177 | 92.5 |
| **0.49** | 3.3 | 19.2 | 0 | 0.164 | 99.9 |
| **0.5** | 3.5 | 19.8 | 0 | 0.152 | 107.8 |
| **0.51** | 3.7 | 20.4 | 0 | 0.141 | 116 |
| **0.52** | 3.9 | 21 | 0 | 0.131 | 124.6 |
| **0.53** | 4.1 | 21.6 | 0 | 0.122 | 133.7 |
| **0.54** | 4.3 | 22.2 | 0 | 0.114 | 143.1 |
| **0.55** | 4.6 | 22.8 | 0 | 0.106 | 153 |
| **0.56** | 4.8 | 23.3 | 0 | 0.099 | 163.3 |
| **0.57** | 5 | 23.9 | 0 | 0.093 | 174 |
| **0.58** | 5.3 | 24.5 | 0 | 0.087 | 185.2 |
| **0.59** | 5.5 | 25.1 | 0 | 0.081 | 196.8 |
| **0.6** | 5.8 | 25.7 | 0 | 0.076 | 208.9 |
| **0.61** | 6 | 26.3 | 0 | 0.072 | 221.4 |
| **0.62** | 6.3 | 26.9 | 0 | 0.068 | 234.5 |
| **0.63** | 6.6 | 27.5 | 0 | 0.064 | 248 |
| **0.64** | 6.9 | 28.1 | 0 | 0.06 | 261.9 |
| **0.65** | 7.1 | 28.7 | 0 | 0.057 | 276.4 |
| **0.66** | 7.4 | 29.3 | 0 | 0.054 | 291.4 |
| **0.67** | 7.7 | 29.8 | 0 | 0.051 | 306.9 |
| **EXCEEDS TOP OF CURB** | | | | | |

## Street Flow Table

Street Half Width = 32’

Curb Type = A2-8”

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.68** | 8 | 30.4 | 0.5 | 0.049 | 319.8 |
| **0.69** | 8.3 | 31 | 1 | 0.047 | 333.5 |
| **0.7** | 8.7 | 31.6 | 1.4 | 0.046 | 347.8 |
| **EXCEEDS CROWN** | | | | | |
| **0.71** | 18 | 64 | 1.9 | 0.044 | 728.6 |
| **0.72** | 18.7 | 64 | 2.4 | 0.041 | 768.1 |
| **0.73** | 19.4 | 64 | 2.9 | 0.039 | 808.8 |
| **0.74** | 20.1 | 64 | 3.4 | 0.037 | 850.6 |
| **0.75** | 20.8 | 64 | 3.8 | 0.035 | 893.5 |
| **0.76** | 21.5 | 64 | 4.3 | 0.033 | 937.6 |
| **0.77** | 22.2 | 64 | 4.8 | 0.031 | 982.8 |
| **0.78** | 23 | 64 | 5.3 | 0.03 | 1029.1 |
| **0.79** | 23.7 | 64 | 5.8 | 0.028 | 1076.6 |
| **0.8** | 24.5 | 64 | 6.2 | 0.027 | 1125.3 |
| **0.81** | 25.3 | 64 | 6.7 | 0.025 | 1175.2 |
| **0.82** | 26 | 64 | 7.2 | 0.024 | 1226.3 |
| **0.83** | 26.8 | 64 | 7.7 | 0.023 | 1278.5 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Table

Street Half Width = 42’
Curb Type = A2-8”

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 2.1 | 0 | 2.055 | 4.6 |
| **0.21** | 0.2 | 2.7 | 0 | 2.223 | 4.7 |
| **0.22** | 0.3 | 3.3 | 0 | 2.239 | 5 |
| **0.23** | 0.3 | 3.8 | 0 | 2.153 | 5.5 |
| **0.24** | 0.3 | 4.4 | 0 | 2.007 | 6.2 |
| **0.25** | 0.4 | 5 | 0 | 1.835 | 7 |
| **0.26** | 0.5 | 5.6 | 0 | 1.656 | 8.1 |
| **0.27** | 0.5 | 6.2 | 0 | 1.484 | 9.3 |
| **0.28** | 0.6 | 6.8 | 0 | 1.323 | 10.7 |
| **0.29** | 0.6 | 7.4 | 0 | 1.177 | 12.3 |
| **0.3** | 0.7 | 8 | 0 | 1.047 | 14.1 |
| **0.31** | 0.8 | 8.6 | 0 | 0.931 | 16.1 |
| **0.32** | 0.9 | 9.2 | 0 | 0.83 | 18.4 |
| **0.33** | 1 | 9.8 | 0 | 0.74 | 20.9 |
| **0.34** | 1.1 | 10.3 | 0 | 0.662 | 23.6 |
| **0.35** | 1.2 | 10.9 | 0 | 0.594 | 26.6 |
| **0.36** | 1.3 | 11.5 | 0 | 0.533 | 29.8 |
| **0.37** | 1.4 | 12.1 | 0 | 0.48 | 33.4 |
| **0.38** | 1.5 | 12.7 | 0 | 0.434 | 37.2 |
| **0.39** | 1.7 | 13.3 | 0 | 0.393 | 41.2 |
| **0.4** | 1.8 | 13.9 | 0 | 0.356 | 45.6 |
| **0.41** | 2 | 14.5 | 0 | 0.324 | 50.3 |
| **0.42** | 2.1 | 15.1 | 0 | 0.296 | 55.3 |
| **0.43** | 2.3 | 15.7 | 0 | 0.27 | 60.7 |
| **0.44** | 2.4 | 16.3 | 0 | 0.247 | 66.3 |
| **0.45** | 2.6 | 16.8 | 0 | 0.227 | 72.3 |
| **0.46** | 2.8 | 17.4 | 0 | 0.209 | 78.7 |
| **0.47** | 2.9 | 18 | 0 | 0.192 | 85.4 |
| **0.48** | 3.1 | 18.6 | 0 | 0.177 | 92.5 |
| **0.49** | 3.3 | 19.2 | 0 | 0.164 | 99.9 |
| **0.5** | 3.5 | 19.8 | 0 | 0.152 | 107.8 |
| **0.51** | 3.7 | 20.4 | 0 | 0.141 | 116 |
| **0.52** | 3.9 | 21 | 0 | 0.131 | 124.6 |
| **0.53** | 4.1 | 21.6 | 0 | 0.122 | 133.7 |
| **0.54** | 4.3 | 22.2 | 0 | 0.114 | 143.1 |
| **0.55** | 4.6 | 22.8 | 0 | 0.106 | 153 |
| **0.56** | 4.8 | 23.3 | 0 | 0.099 | 163.3 |
| **0.57** | 5 | 23.9 | 0 | 0.093 | 174 |
| **0.58** | 5.3 | 24.5 | 0 | 0.087 | 185.2 |
| **0.59** | 5.5 | 25.1 | 0 | 0.081 | 196.8 |
| **0.6** | 5.8 | 25.7 | 0 | 0.076 | 208.9 |
| **0.61** | 6 | 26.3 | 0 | 0.072 | 221.4 |
| **0.62** | 6.3 | 26.9 | 0 | 0.068 | 234.5 |
| **0.63** | 6.6 | 27.5 | 0 | 0.064 | 248 |
| **0.64** | 6.9 | 28.1 | 0 | 0.06 | 261.9 |
| **0.65** | 7.1 | 28.7 | 0 | 0.057 | 276.4 |
| **0.66** | 7.4 | 29.3 | 0 | 0.054 | 291.4 |
| **0.67** | 7.7 | 29.8 | 0 | 0.051 | 306.9 |
| **EXCEEDS TOP OF CURB** | | | | | |

## Street Flow Table

Street Half Width = 42’

Curb Type = A2-8”

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.68** | 8 | 30.4 | 0.5 | 0.049 | 319.8 |
| **0.69** | 8.3 | 31 | 1 | 0.047 | 333.5 |
| **0.7** | 8.7 | 31.6 | 1.4 | 0.046 | 347.8 |
| **0.71** | 9 | 32.2 | 1.9 | 0.044 | 362.9 |
| **0.72** | 9.3 | 32.8 | 2.4 | 0.042 | 378.8 |
| **0.73** | 9.7 | 33.4 | 2.9 | 0.041 | 395.4 |
| **0.74** | 10.1 | 34 | 3.4 | 0.039 | 412.7 |
| **0.75** | 10.4 | 34.5 | 3.8 | 0.038 | 430.8 |
| **0.76** | 10.8 | 35.1 | 4.3 | 0.036 | 449.7 |
| **0.77** | 11.2 | 35.7 | 4.8 | 0.035 | 469.4 |
| **0.78** | 11.6 | 36.3 | 5.3 | 0.033 | 489.8 |
| **0.79** | 12.1 | 36.9 | 5.8 | 0.032 | 511.1 |
| **0.8** | 12.5 | 37.5 | 6.2 | 0.031 | 533.2 |
| **0.81** | 12.9 | 38.1 | 6.7 | 0.03 | 556.1 |
| **0.82** | 13.4 | 38.7 | 7.2 | 0.029 | 579.9 |
| **0.83** | 13.9 | 39.3 | 7.7 | 0.027 | 604.5 |
| **0.84** | 14.3 | 39.8 | 8.2 | 0.026 | 630 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Table

Street Half Width = 51'
Curb Type = A2-8"

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 2.1 | 0 | 2.055 | 4.6 |
| **0.21** | 0.2 | 2.7 | 0 | 2.223 | 4.7 |
| **0.22** | 0.3 | 3.3 | 0 | 2.239 | 5 |
| **0.23** | 0.3 | 3.8 | 0 | 2.153 | 5.5 |
| **0.24** | 0.3 | 4.4 | 0 | 2.007 | 6.2 |
| **0.25** | 0.4 | 5 | 0 | 1.835 | 7 |
| **0.26** | 0.5 | 5.6 | 0 | 1.656 | 8.1 |
| **0.27** | 0.5 | 6.2 | 0 | 1.484 | 9.3 |
| **0.28** | 0.6 | 6.8 | 0 | 1.323 | 10.7 |
| **0.29** | 0.6 | 7.4 | 0 | 1.177 | 12.3 |
| **0.3** | 0.7 | 8 | 0 | 1.047 | 14.1 |
| **0.31** | 0.8 | 8.6 | 0 | 0.931 | 16.1 |
| **0.32** | 0.9 | 9.2 | 0 | 0.83 | 18.4 |
| **0.33** | 1 | 9.8 | 0 | 0.74 | 20.9 |
| **0.34** | 1.1 | 10.3 | 0 | 0.662 | 23.6 |
| **0.35** | 1.2 | 10.9 | 0 | 0.594 | 26.6 |
| **0.36** | 1.3 | 11.5 | 0 | 0.533 | 29.8 |
| **0.37** | 1.4 | 12.1 | 0 | 0.48 | 33.4 |
| **0.38** | 1.5 | 12.7 | 0 | 0.434 | 37.2 |
| **0.39** | 1.7 | 13.3 | 0 | 0.393 | 41.2 |
| **0.4** | 1.8 | 13.9 | 0 | 0.356 | 45.6 |
| **0.41** | 2 | 14.5 | 0 | 0.324 | 50.3 |
| **0.42** | 2.1 | 15.1 | 0 | 0.296 | 55.3 |
| **0.43** | 2.3 | 15.7 | 0 | 0.27 | 60.7 |
| **0.44** | 2.4 | 16.3 | 0 | 0.247 | 66.3 |
| **0.45** | 2.6 | 16.8 | 0 | 0.227 | 72.3 |
| **0.46** | 2.8 | 17.4 | 0 | 0.209 | 78.7 |
| **0.47** | 2.9 | 18 | 0 | 0.192 | 85.4 |
| **0.48** | 3.1 | 18.6 | 0 | 0.177 | 92.5 |
| **0.49** | 3.3 | 19.2 | 0 | 0.164 | 99.9 |
| **0.5** | 3.5 | 19.8 | 0 | 0.152 | 107.8 |
| **0.51** | 3.7 | 20.4 | 0 | 0.141 | 116 |
| **0.52** | 3.9 | 21 | 0 | 0.131 | 124.6 |
| **0.53** | 4.1 | 21.6 | 0 | 0.122 | 133.7 |
| **0.54** | 4.3 | 22.2 | 0 | 0.114 | 143.1 |
| **0.55** | 4.6 | 22.8 | 0 | 0.106 | 153 |
| **0.56** | 4.8 | 23.3 | 0 | 0.099 | 163.3 |
| **0.57** | 5 | 23.9 | 0 | 0.093 | 174 |
| **0.58** | 5.3 | 24.5 | 0 | 0.087 | 185.2 |
| **0.59** | 5.5 | 25.1 | 0 | 0.081 | 196.8 |
| **0.6** | 5.8 | 25.7 | 0 | 0.076 | 208.9 |
| **0.61** | 6 | 26.3 | 0 | 0.072 | 221.4 |
| **0.62** | 6.3 | 26.9 | 0 | 0.068 | 234.5 |
| **0.63** | 6.6 | 27.5 | 0 | 0.064 | 248 |
| **0.64** | 6.9 | 28.1 | 0 | 0.06 | 261.9 |
| **0.65** | 7.1 | 28.7 | 0 | 0.057 | 276.4 |
| **0.66** | 7.4 | 29.3 | 0 | 0.054 | 291.4 |
| **0.67** | 7.7 | 29.8 | 0 | 0.051 | 306.9 |
| **EXCEEDS TOP OF CURB** | | | | | |

Street Half Width = 51’

Curb Type = A2-8”

## Street Flow Table

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.68** | 8 | 30.4 | 0.5 | 0.049 | 319.8 |
| **0.69** | 8.3 | 31 | 1 | 0.047 | 333.5 |
| **0.7** | 8.7 | 31.6 | 1.4 | 0.046 | 347.8 |
| **0.71** | 9 | 32.2 | 1.9 | 0.044 | 362.9 |
| **0.72** | 9.3 | 32.8 | 2.4 | 0.042 | 378.8 |
| **0.73** | 9.7 | 33.4 | 2.9 | 0.041 | 395.4 |
| **0.74** | 10.1 | 34 | 3.4 | 0.039 | 412.7 |
| **0.75** | 10.4 | 34.5 | 3.8 | 0.038 | 430.8 |
| **0.76** | 10.8 | 35.1 | 4.3 | 0.036 | 449.7 |
| **0.77** | 11.2 | 35.7 | 4.8 | 0.035 | 469.4 |
| **0.78** | 11.6 | 36.3 | 5.3 | 0.033 | 489.8 |
| **0.79** | 12.1 | 36.9 | 5.8 | 0.032 | 511.1 |
| **0.8** | 12.5 | 37.5 | 6.2 | 0.031 | 533.2 |
| **0.81** | 12.9 | 38.1 | 6.7 | 0.03 | 556.1 |
| **0.82** | 13.4 | 38.7 | 7.2 | 0.029 | 579.9 |
| **0.83** | 13.9 | 39.3 | 7.7 | 0.027 | 604.5 |
| **0.84** | 14.3 | 39.8 | 8.2 | 0.026 | 630 |
| **0.85** | 14.8 | 40.4 | 8.6 | 0.025 | 656.3 |
| **0.86** | 15.3 | 41 | 9.1 | 0.024 | 683.6 |
| **EXCEEDS R/W** | | | | | |

## A.2 Rolled Curb Type

## Street Flow Tables

Street Half Width = 14'
Curb Type = Rolled

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 3.6 | 0 | 4.317 | 3.4 |
| **0.21** | 0.3 | 4.2 | 0 | 3.739 | 4.2 |
| **0.22** | 0.3 | 4.8 | 0 | 3.208 | 5.1 |
| **0.23** | 0.4 | 5.3 | 0 | 2.744 | 6.1 |
| **0.24** | 0.5 | 5.9 | 0 | 2.346 | 7.4 |
| **0.25** | 0.5 | 6.5 | 0 | 2.011 | 8.9 |
| **0.26** | 0.6 | 7.1 | 0 | 1.73 | 10.5 |
| **0.27** | 0.7 | 7.7 | 0 | 1.494 | 12.4 |
| **0.28** | 0.8 | 8.3 | 0 | 1.296 | 14.5 |
| **0.29** | 0.9 | 8.9 | 0 | 1.129 | 16.9 |
| **0.3** | 1 | 9.5 | 0 | 0.988 | 19.5 |
| **0.31** | 1.1 | 10 | 0 | 0.868 | 22.3 |
| **0.32** | 1.2 | 10.6 | 0 | 0.767 | 25.5 |
| **0.33** | 1.3 | 11.2 | 0 | 0.679 | 28.9 |
| **0.34** | 1.4 | 11.8 | 0 | 0.604 | 32.6 |
| **0.35** | 1.6 | 12.4 | 0 | 0.54 | 36.6 |
| **0.36** | 1.7 | 13 | 0 | 0.484 | 41 |
| **0.37** | 1.9 | 13.6 | 0 | 0.435 | 45.6 |
| **EXCEEDS CROWN** | | | | | |
| **0.38** | 4 | 28 | 0 | 0.387 | 101.9 |
| **0.39** | 4.3 | 28 | 0 | 0.334 | 115.1 |
| **0.4** | 4.6 | 28 | 0 | 0.291 | 128.8 |
| **0.41** | 4.9 | 28 | 0 | 0.255 | 143.2 |
| **0.42** | 5.3 | 28 | 0 | 0.225 | 158.2 |
| **0.43** | 5.6 | 28 | 0 | 0.199 | 173.8 |
| **0.44** | 5.9 | 28 | 0 | 0.178 | 190 |
| **0.45** | 6.2 | 28 | 0 | 0.159 | 206.8 |
| **0.46** | 6.5 | 28 | 0 | 0.143 | 224.1 |
| **0.47** | 6.8 | 28 | 0 | 0.129 | 242 |
| **0.48** | 7.1 | 28 | 0 | 0.117 | 260.5 |
| **0.49** | 7.4 | 28 | 0 | 0.106 | 279.5 |
| **0.5** | 7.8 | 28 | 0 | 0.097 | 299.1 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 8.1 | 28 | 0.5 | 0.099 | 301.8 |
| **0.52** | 8.4 | 28 | 1 | 0.1 | 307.2 |
| **0.53** | 8.8 | 28 | 1.4 | 0.1 | 313.5 |
| **0.54** | 9.1 | 28 | 1.9 | 0.1 | 320.6 |
| **0.55** | 9.5 | 28 | 2.4 | 0.099 | 328.6 |
| **0.56** | 9.9 | 28 | 2.9 | 0.098 | 337.2 |
| **0.57** | 10.2 | 28 | 3.4 | 0.097 | 346.5 |
| **0.58** | 10.6 | 28 | 3.8 | 0.095 | 356.4 |
| **0.59** | 11 | 28 | 4.3 | 0.093 | 366.9 |
| **0.6** | 11.4 | 28 | 4.8 | 0.092 | 378 |
| **0.61** | 11.9 | 28 | 5.3 | 0.09 | 389.6 |
| **0.62** | 12.3 | 28 | 5.8 | 0.088 | 401.9 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Tables

Street Half Width = 15'
Curb Type = Rolled

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 3.6 | 0 | 4.317 | 3.4 |
| **0.21** | 0.3 | 4.2 | 0 | 3.739 | 4.2 |
| **0.22** | 0.3 | 4.8 | 0 | 3.208 | 5.1 |
| **0.23** | 0.4 | 5.3 | 0 | 2.744 | 6.1 |
| **0.24** | 0.5 | 5.9 | 0 | 2.346 | 7.4 |
| **0.25** | 0.5 | 6.5 | 0 | 2.011 | 8.9 |
| **0.26** | 0.6 | 7.1 | 0 | 1.73 | 10.5 |
| **0.27** | 0.7 | 7.7 | 0 | 1.494 | 12.4 |
| **0.28** | 0.8 | 8.3 | 0 | 1.296 | 14.5 |
| **0.29** | 0.9 | 8.9 | 0 | 1.129 | 16.9 |
| **0.3** | 1 | 9.5 | 0 | 0.988 | 19.5 |
| **0.31** | 1.1 | 10 | 0 | 0.868 | 22.3 |
| **0.32** | 1.2 | 10.6 | 0 | 0.767 | 25.5 |
| **0.33** | 1.3 | 11.2 | 0 | 0.679 | 28.9 |
| **0.34** | 1.4 | 11.8 | 0 | 0.604 | 32.6 |
| **0.35** | 1.6 | 12.4 | 0 | 0.54 | 36.6 |
| **0.36** | 1.7 | 13 | 0 | 0.484 | 41 |
| **0.37** | 1.9 | 13.6 | 0 | 0.435 | 45.6 |
| **0.38** | 2 | 14.2 | 0 | 0.392 | 50.6 |
| **0.39** | 2.2 | 14.7 | 0 | 0.355 | 56 |
| **EXCEEDS CROWN** | | | | | |
| **0.4** | 4.7 | 30 | 0 | 0.313 | 124.9 |
| **0.41** | 5 | 30 | 0 | 0.273 | 139.7 |
| **0.42** | 5.3 | 30 | 0 | 0.24 | 155.1 |
| **0.43** | 5.7 | 30 | 0 | 0.212 | 171.2 |
| **0.44** | 6 | 30 | 0 | 0.188 | 188 |
| **0.45** | 6.3 | 30 | 0 | 0.168 | 205.4 |
| **0.46** | 6.6 | 30 | 0 | 0.151 | 223.4 |
| **0.47** | 7 | 30 | 0 | 0.136 | 242 |
| **0.48** | 7.3 | 30 | 0 | 0.123 | 261.3 |
| **0.49** | 7.7 | 30 | 0 | 0.111 | 281.1 |
| **0.5** | 8 | 30 | 0 | 0.101 | 301.5 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 8.3 | 30 | 0.5 | 0.103 | 305.9 |
| **0.52** | 8.7 | 30 | 1 | 0.103 | 312.7 |
| **0.53** | 9.1 | 30 | 1.4 | 0.102 | 320.4 |
| **0.54** | 9.4 | 30 | 1.9 | 0.101 | 328.8 |
| **0.55** | 9.8 | 30 | 2.4 | 0.1 | 338 |
| **0.56** | 10.2 | 30 | 2.9 | 0.099 | 347.8 |
| **0.57** | 10.6 | 30 | 3.4 | 0.097 | 358.2 |
| **0.58** | 11 | 30 | 3.8 | 0.095 | 369.3 |
| **0.59** | 11.4 | 30 | 4.3 | 0.093 | 380.9 |
| **0.6** | 11.9 | 30 | 4.8 | 0.091 | 393.1 |
| **0.61** | 12.3 | 30 | 5.3 | 0.089 | 405.8 |
| **0.62** | 12.8 | 30 | 5.8 | 0.087 | 419.1 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Tables

Street Half Width = 16’
Curb Type = Rolled

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 3.6 | 0 | 4.317 | 3.4 |
| **0.21** | 0.3 | 4.2 | 0 | 3.739 | 4.2 |
| **0.22** | 0.3 | 4.8 | 0 | 3.208 | 5.1 |
| **0.23** | 0.4 | 5.3 | 0 | 2.744 | 6.1 |
| **0.24** | 0.5 | 5.9 | 0 | 2.346 | 7.4 |
| **0.25** | 0.5 | 6.5 | 0 | 2.011 | 8.9 |
| **0.26** | 0.6 | 7.1 | 0 | 1.73 | 10.5 |
| **0.27** | 0.7 | 7.7 | 0 | 1.494 | 12.4 |
| **0.28** | 0.8 | 8.3 | 0 | 1.296 | 14.5 |
| **0.29** | 0.9 | 8.9 | 0 | 1.129 | 16.9 |
| **0.3** | 1 | 9.5 | 0 | 0.988 | 19.5 |
| **0.31** | 1.1 | 10 | 0 | 0.868 | 22.3 |
| **0.32** | 1.2 | 10.6 | 0 | 0.767 | 25.5 |
| **0.33** | 1.3 | 11.2 | 0 | 0.679 | 28.9 |
| **0.34** | 1.4 | 11.8 | 0 | 0.604 | 32.6 |
| **0.35** | 1.6 | 12.4 | 0 | 0.54 | 36.6 |
| **0.36** | 1.7 | 13 | 0 | 0.484 | 41 |
| **0.37** | 1.9 | 13.6 | 0 | 0.435 | 45.6 |
| **0.38** | 2 | 14.2 | 0 | 0.392 | 50.6 |
| **0.39** | 2.2 | 14.7 | 0 | 0.355 | 56 |
| **0.4** | 2.3 | 15.3 | 0 | 0.322 | 61.6 |
| **0.41** | 2.5 | 15.9 | 0 | 0.293 | 67.7 |
| **EXCEEDS CROWN** | | | | | |
| **0.42** | 5.4 | 32 | 0 | 0.257 | 150.8 |
| **0.43** | 5.7 | 32 | 0 | 0.226 | 167.4 |
| **0.44** | 6.1 | 32 | 0 | 0.2 | 184.6 |
| **0.45** | 6.4 | 32 | 0 | 0.178 | 202.6 |
| **0.46** | 6.8 | 32 | 0 | 0.159 | 221.2 |
| **0.47** | 7.1 | 32 | 0 | 0.143 | 240.4 |
| **0.48** | 7.5 | 32 | 0 | 0.129 | 260.3 |
| **0.49** | 7.8 | 32 | 0 | 0.116 | 280.9 |
| **0.5** | 8.2 | 32 | 0 | 0.106 | 302.1 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 8.6 | 32 | 0.5 | 0.107 | 308.1 |
| **0.52** | 8.9 | 32 | 1 | 0.106 | 316.4 |
| **0.53** | 9.3 | 32 | 1.4 | 0.105 | 325.4 |
| **0.54** | 9.7 | 32 | 1.9 | 0.103 | 335.1 |
| **0.55** | 10.1 | 32 | 2.4 | 0.102 | 345.5 |
| **0.56** | 10.5 | 32 | 2.9 | 0.1 | 356.5 |
| **0.57** | 10.9 | 32 | 3.4 | 0.098 | 368.1 |
| **0.58** | 11.4 | 32 | 3.8 | 0.096 | 380.3 |
| **0.59** | 11.8 | 32 | 4.3 | 0.093 | 393 |
| **0.6** | 12.3 | 32 | 4.8 | 0.091 | 406.3 |
| **0.61** | 12.7 | 32 | 5.3 | 0.089 | 420.2 |
| **0.62** | 13.2 | 32 | 5.8 | 0.086 | 434.6 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Tables

Street Half Width = 17'
Curb Type = Rolled

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 3.6 | 0 | 4.317 | 3.4 |
| **0.21** | 0.3 | 4.2 | 0 | 3.739 | 4.2 |
| **0.22** | 0.3 | 4.8 | 0 | 3.208 | 5.1 |
| **0.23** | 0.4 | 5.3 | 0 | 2.744 | 6.1 |
| **0.24** | 0.5 | 5.9 | 0 | 2.346 | 7.4 |
| **0.25** | 0.5 | 6.5 | 0 | 2.011 | 8.9 |
| **0.26** | 0.6 | 7.1 | 0 | 1.73 | 10.5 |
| **0.27** | 0.7 | 7.7 | 0 | 1.494 | 12.4 |
| **0.28** | 0.8 | 8.3 | 0 | 1.296 | 14.5 |
| **0.29** | 0.9 | 8.9 | 0 | 1.129 | 16.9 |
| **0.3** | 1 | 9.5 | 0 | 0.988 | 19.5 |
| **0.31** | 1.1 | 10 | 0 | 0.868 | 22.3 |
| **0.32** | 1.2 | 10.6 | 0 | 0.767 | 25.5 |
| **0.33** | 1.3 | 11.2 | 0 | 0.679 | 28.9 |
| **0.34** | 1.4 | 11.8 | 0 | 0.604 | 32.6 |
| **0.35** | 1.6 | 12.4 | 0 | 0.54 | 36.6 |
| **0.36** | 1.7 | 13 | 0 | 0.484 | 41 |
| **0.37** | 1.9 | 13.6 | 0 | 0.435 | 45.6 |
| **0.38** | 2 | 14.2 | 0 | 0.392 | 50.6 |
| **0.39** | 2.2 | 14.7 | 0 | 0.355 | 56 |
| **0.4** | 2.3 | 15.3 | 0 | 0.322 | 61.6 |
| **0.41** | 2.5 | 15.9 | 0 | 0.293 | 67.7 |
| **0.42** | 2.7 | 16.5 | 0 | 0.267 | 74.1 |
| **EXCEEDS CROWN** | | | | | |
| **0.43** | 5.7 | 34 | 0 | 0.242 | 162.3 |
| **0.44** | 6.1 | 34 | 0 | 0.213 | 180 |
| **0.45** | 6.5 | 34 | 0 | 0.189 | 198.4 |
| **0.46** | 6.8 | 34 | 0 | 0.168 | 217.5 |
| **0.47** | 7.2 | 34 | 0 | 0.151 | 237.3 |
| **0.48** | 7.6 | 34 | 0 | 0.135 | 257.9 |
| **0.49** | 8 | 34 | 0 | 0.122 | 279.1 |
| **0.5** | 8.3 | 34 | 0 | 0.111 | 301 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 8.7 | 34 | 0.5 | 0.111 | 308.5 |
| **0.52** | 9.1 | 34 | 1 | 0.109 | 318.2 |
| **0.53** | 9.5 | 34 | 1.4 | 0.108 | 328.5 |
| **0.54** | 9.9 | 34 | 1.9 | 0.106 | 339.5 |
| **0.55** | 10.4 | 34 | 2.4 | 0.104 | 351.1 |
| **0.56** | 10.8 | 34 | 2.9 | 0.101 | 363.3 |
| **0.57** | 11.2 | 34 | 3.4 | 0.099 | 376.1 |
| **0.58** | 11.7 | 34 | 3.8 | 0.096 | 389.4 |
| **0.59** | 12.2 | 34 | 4.3 | 0.094 | 403.3 |
| **0.6** | 12.6 | 34 | 4.8 | 0.091 | 417.7 |
| **0.61** | 13.1 | 34 | 5.3 | 0.089 | 432.6 |
| **0.62** | 13.6 | 34 | 5.8 | 0.086 | 448.1 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Tables

Street Half Width = 18’
Curb Type = Rolled

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 3.6 | 0 | 4.317 | 3.4 |
| **0.21** | 0.3 | 4.2 | 0 | 3.739 | 4.2 |
| **0.22** | 0.3 | 4.8 | 0 | 3.208 | 5.1 |
| **0.23** | 0.4 | 5.3 | 0 | 2.744 | 6.1 |
| **0.24** | 0.5 | 5.9 | 0 | 2.346 | 7.4 |
| **0.25** | 0.5 | 6.5 | 0 | 2.011 | 8.9 |
| **0.26** | 0.6 | 7.1 | 0 | 1.73 | 10.5 |
| **0.27** | 0.7 | 7.7 | 0 | 1.494 | 12.4 |
| **0.28** | 0.8 | 8.3 | 0 | 1.296 | 14.5 |
| **0.29** | 0.9 | 8.9 | 0 | 1.129 | 16.9 |
| **0.3** | 1 | 9.5 | 0 | 0.988 | 19.5 |
| **0.31** | 1.1 | 10 | 0 | 0.868 | 22.3 |
| **0.32** | 1.2 | 10.6 | 0 | 0.767 | 25.5 |
| **0.33** | 1.3 | 11.2 | 0 | 0.679 | 28.9 |
| **0.34** | 1.4 | 11.8 | 0 | 0.604 | 32.6 |
| **0.35** | 1.6 | 12.4 | 0 | 0.54 | 36.6 |
| **0.36** | 1.7 | 13 | 0 | 0.484 | 41 |
| **0.37** | 1.9 | 13.6 | 0 | 0.435 | 45.6 |
| **0.38** | 2 | 14.2 | 0 | 0.392 | 50.6 |
| **0.39** | 2.2 | 14.7 | 0 | 0.355 | 56 |
| **0.4** | 2.3 | 15.3 | 0 | 0.322 | 61.6 |
| **0.41** | 2.5 | 15.9 | 0 | 0.293 | 67.7 |
| **0.42** | 2.7 | 16.5 | 0 | 0.267 | 74.1 |
| **0.43** | 2.9 | 17.1 | 0 | 0.244 | 80.9 |
| **0.44** | 3.1 | 17.7 | 0 | 0.224 | 88 |
| **EXCEEDS CROWN** | | | | | |
| **0.45** | 6.5 | 36 | 0 | 0.202 | 193 |
| **0.46** | 6.9 | 36 | 0 | 0.179 | 212.5 |
| **0.47** | 7.3 | 36 | 0 | 0.16 | 232.8 |
| **0.48** | 7.7 | 36 | 0 | 0.143 | 253.9 |
| **0.49** | 8.1 | 36 | 0 | 0.129 | 275.7 |
| **0.5** | 8.5 | 36 | 0 | 0.116 | 298.2 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 8.9 | 36 | 0.5 | 0.116 | 307.3 |
| **0.52** | 9.3 | 36 | 1 | 0.113 | 318.3 |
| **0.53** | 9.7 | 36 | 1.4 | 0.111 | 329.9 |
| **0.54** | 10.1 | 36 | 1.9 | 0.109 | 342.1 |
| **0.55** | 10.6 | 36 | 2.4 | 0.106 | 354.9 |
| **0.56** | 11 | 36 | 2.9 | 0.103 | 368.3 |
| **0.57** | 11.5 | 36 | 3.4 | 0.1 | 382.2 |
| **0.58** | 12 | 36 | 3.8 | 0.098 | 396.7 |
| **0.59** | 12.5 | 36 | 4.3 | 0.095 | 411.7 |
| **0.6** | 13 | 36 | 4.8 | 0.092 | 427.2 |
| **0.61** | 13.5 | 36 | 5.3 | 0.089 | 443.2 |
| **0.62** | 14 | 36 | 5.8 | 0.086 | 459.8 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Tables

Street Half Width = 19'
Curb Type = Rolled

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 3.6 | 0 | 4.317 | 3.4 |
| **0.21** | 0.3 | 4.2 | 0 | 3.739 | 4.2 |
| **0.22** | 0.3 | 4.8 | 0 | 3.208 | 5.1 |
| **0.23** | 0.4 | 5.3 | 0 | 2.744 | 6.1 |
| **0.24** | 0.5 | 5.9 | 0 | 2.346 | 7.4 |
| **0.25** | 0.5 | 6.5 | 0 | 2.011 | 8.9 |
| **0.26** | 0.6 | 7.1 | 0 | 1.73 | 10.5 |
| **0.27** | 0.7 | 7.7 | 0 | 1.494 | 12.4 |
| **0.28** | 0.8 | 8.3 | 0 | 1.296 | 14.5 |
| **0.29** | 0.9 | 8.9 | 0 | 1.129 | 16.9 |
| **0.3** | 1 | 9.5 | 0 | 0.988 | 19.5 |
| **0.31** | 1.1 | 10 | 0 | 0.868 | 22.3 |
| **0.32** | 1.2 | 10.6 | 0 | 0.767 | 25.5 |
| **0.33** | 1.3 | 11.2 | 0 | 0.679 | 28.9 |
| **0.34** | 1.4 | 11.8 | 0 | 0.604 | 32.6 |
| **0.35** | 1.6 | 12.4 | 0 | 0.54 | 36.6 |
| **0.36** | 1.7 | 13 | 0 | 0.484 | 41 |
| **0.37** | 1.9 | 13.6 | 0 | 0.435 | 45.6 |
| **0.38** | 2 | 14.2 | 0 | 0.392 | 50.6 |
| **0.39** | 2.2 | 14.7 | 0 | 0.355 | 56 |
| **0.4** | 2.3 | 15.3 | 0 | 0.322 | 61.6 |
| **0.41** | 2.5 | 15.9 | 0 | 0.293 | 67.7 |
| **0.42** | 2.7 | 16.5 | 0 | 0.267 | 74.1 |
| **0.43** | 2.9 | 17.1 | 0 | 0.244 | 80.9 |
| **0.44** | 3.1 | 17.7 | 0 | 0.224 | 88 |
| **0.45** | 3.2 | 18.3 | 0 | 0.205 | 95.6 |
| **0.46** | 3.5 | 18.9 | 0 | 0.189 | 103.6 |
| **EXCEEDS CROWN** | | | | | |
| **0.47** | 7.3 | 38 | 0 | 0.169 | 227 |
| **0.48** | 7.7 | 38 | 0 | 0.151 | 248.6 |
| **0.49** | 8.1 | 38 | 0 | 0.136 | 270.9 |
| **0.5** | 8.6 | 38 | 0 | 0.122 | 294 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 9 | 38 | 0.5 | 0.121 | 304.4 |
| **0.52** | 9.4 | 38 | 1 | 0.118 | 316.7 |
| **0.53** | 9.9 | 38 | 1.4 | 0.115 | 329.6 |
| **0.54** | 10.3 | 38 | 1.9 | 0.112 | 343 |
| **0.55** | 10.8 | 38 | 2.4 | 0.109 | 357 |
| **0.56** | 11.3 | 38 | 2.9 | 0.105 | 371.5 |
| **0.57** | 11.7 | 38 | 3.4 | 0.102 | 386.5 |
| **0.58** | 12.2 | 38 | 3.8 | 0.099 | 402.1 |
| **0.59** | 12.7 | 38 | 4.3 | 0.096 | 418.2 |
| **0.6** | 13.2 | 38 | 4.8 | 0.093 | 434.8 |
| **0.61** | 13.8 | 38 | 5.3 | 0.09 | 452 |
| **0.62** | 14.3 | 38 | 5.8 | 0.087 | 469.7 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Tables

Street Half Width = 20’
Curb Type = Rolled

| **Flow Depth (ft)** | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 3.6 | 0 | 4.317 | 3.4 |
| **0.21** | 0.3 | 4.2 | 0 | 3.739 | 4.2 |
| **0.22** | 0.3 | 4.8 | 0 | 3.208 | 5.1 |
| **0.23** | 0.4 | 5.3 | 0 | 2.744 | 6.1 |
| **0.24** | 0.5 | 5.9 | 0 | 2.346 | 7.4 |
| **0.25** | 0.5 | 6.5 | 0 | 2.011 | 8.9 |
| **0.26** | 0.6 | 7.1 | 0 | 1.73 | 10.5 |
| **0.27** | 0.7 | 7.7 | 0 | 1.494 | 12.4 |
| **0.28** | 0.8 | 8.3 | 0 | 1.296 | 14.5 |
| **0.29** | 0.9 | 8.9 | 0 | 1.129 | 16.9 |
| **0.3** | 1 | 9.5 | 0 | 0.988 | 19.5 |
| **0.31** | 1.1 | 10 | 0 | 0.868 | 22.3 |
| **0.32** | 1.2 | 10.6 | 0 | 0.767 | 25.5 |
| **0.33** | 1.3 | 11.2 | 0 | 0.679 | 28.9 |
| **0.34** | 1.4 | 11.8 | 0 | 0.604 | 32.6 |
| **0.35** | 1.6 | 12.4 | 0 | 0.54 | 36.6 |
| **0.36** | 1.7 | 13 | 0 | 0.484 | 41 |
| **0.37** | 1.9 | 13.6 | 0 | 0.435 | 45.6 |
| **0.38** | 2 | 14.2 | 0 | 0.392 | 50.6 |
| **0.39** | 2.2 | 14.7 | 0 | 0.355 | 56 |
| **0.4** | 2.3 | 15.3 | 0 | 0.322 | 61.6 |
| **0.41** | 2.5 | 15.9 | 0 | 0.293 | 67.7 |
| **0.42** | 2.7 | 16.5 | 0 | 0.267 | 74.1 |
| **0.43** | 2.9 | 17.1 | 0 | 0.244 | 80.9 |
| **0.44** | 3.1 | 17.7 | 0 | 0.224 | 88 |
| **0.45** | 3.2 | 18.3 | 0 | 0.205 | 95.6 |
| **0.46** | 3.5 | 18.9 | 0 | 0.189 | 103.6 |
| **0.47** | 3.7 | 19.5 | 0 | 0.174 | 112 |
| **EXCEEDS CROWN** | | | | | |
| **0.48** | 7.8 | 40 | 0 | 0.16 | 241.9 |
| **0.49** | 8.2 | 40 | 0 | 0.143 | 264.6 |
| **0.5** | 8.6 | 40 | 0 | 0.129 | 288.2 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 9.1 | 40 | 0.5 | 0.126 | 300 |
| **0.52** | 9.5 | 40 | 1 | 0.123 | 313.5 |
| **0.53** | 10 | 40 | 1.4 | 0.119 | 327.6 |
| **0.54** | 10.5 | 40 | 1.9 | 0.115 | 342.2 |
| **0.55** | 10.9 | 40 | 2.4 | 0.112 | 357.3 |
| **0.56** | 11.4 | 40 | 2.9 | 0.108 | 372.9 |
| **0.57** | 11.9 | 40 | 3.4 | 0.104 | 389 |
| **0.58** | 12.5 | 40 | 3.8 | 0.101 | 405.7 |
| **0.59** | 13 | 40 | 4.3 | 0.097 | 422.9 |
| **0.6** | 13.5 | 40 | 4.8 | 0.094 | 440.6 |
| **0.61** | 14 | 40 | 5.3 | 0.091 | 458.9 |
| **0.62** | 14.6 | 40 | 5.8 | 0.087 | 477.6 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Tables

Street Half Width = 21'
Curb Type = Rolled

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 3.6 | 0 | 4.317 | 3.4 |
| **0.21** | 0.3 | 4.2 | 0 | 3.739 | 4.2 |
| **0.22** | 0.3 | 4.8 | 0 | 3.208 | 5.1 |
| **0.23** | 0.4 | 5.3 | 0 | 2.744 | 6.1 |
| **0.24** | 0.5 | 5.9 | 0 | 2.346 | 7.4 |
| **0.25** | 0.5 | 6.5 | 0 | 2.011 | 8.9 |
| **0.26** | 0.6 | 7.1 | 0 | 1.73 | 10.5 |
| **0.27** | 0.7 | 7.7 | 0 | 1.494 | 12.4 |
| **0.28** | 0.8 | 8.3 | 0 | 1.296 | 14.5 |
| **0.29** | 0.9 | 8.9 | 0 | 1.129 | 16.9 |
| **0.3** | 1 | 9.5 | 0 | 0.988 | 19.5 |
| **0.31** | 1.1 | 10 | 0 | 0.868 | 22.3 |
| **0.32** | 1.2 | 10.6 | 0 | 0.767 | 25.5 |
| **0.33** | 1.3 | 11.2 | 0 | 0.679 | 28.9 |
| **0.34** | 1.4 | 11.8 | 0 | 0.604 | 32.6 |
| **0.35** | 1.6 | 12.4 | 0 | 0.54 | 36.6 |
| **0.36** | 1.7 | 13 | 0 | 0.484 | 41 |
| **0.37** | 1.9 | 13.6 | 0 | 0.435 | 45.6 |
| **0.38** | 2 | 14.2 | 0 | 0.392 | 50.6 |
| **0.39** | 2.2 | 14.7 | 0 | 0.355 | 56 |
| **0.4** | 2.3 | 15.3 | 0 | 0.322 | 61.6 |
| **0.41** | 2.5 | 15.9 | 0 | 0.293 | 67.7 |
| **0.42** | 2.7 | 16.5 | 0 | 0.267 | 74.1 |
| **0.43** | 2.9 | 17.1 | 0 | 0.244 | 80.9 |
| **0.44** | 3.1 | 17.7 | 0 | 0.224 | 88 |
| **0.45** | 3.2 | 18.3 | 0 | 0.205 | 95.6 |
| **0.46** | 3.5 | 18.9 | 0 | 0.189 | 103.6 |
| **0.47** | 3.7 | 19.5 | 0 | 0.174 | 112 |
| **0.48** | 3.9 | 20 | 0 | 0.161 | 120.8 |
| **0.49** | 4.1 | 20.6 | 0 | 0.149 | 130 |
| **0.5** | 4.3 | 21.2 | 0 | 0.138 | 139.7 |
| **EXCEEDS CURB AND CROWN** | | | | | |
| **0.51** | 9.7 | 42 | 0.5 | 0.128 | 318 |
| **0.52** | 10.6 | 42 | 1 | 0.121 | 350.9 |
| **0.53** | 11.5 | 42 | 1.4 | 0.114 | 385 |
| **0.54** | 12.4 | 42 | 1.9 | 0.108 | 420.4 |
| **0.55** | 13.4 | 42 | 2.4 | 0.102 | 457.1 |
| **0.56** | 14.4 | 42 | 2.9 | 0.097 | 495.1 |
| **0.57** | 15.4 | 42 | 3.4 | 0.092 | 534.4 |
| **0.58** | 16.4 | 42 | 3.8 | 0.087 | 575 |
| **0.59** | 17.4 | 42 | 4.3 | 0.083 | 617.1 |
| **0.6** | 18.5 | 42 | 4.8 | 0.079 | 660.5 |
| **0.61** | 19.6 | 42 | 5.3 | 0.075 | 705.2 |
| **0.62** | 20.7 | 42 | 5.8 | 0.071 | 751.4 |
| **EXCEEDS R/W** | | | | | |

## Street Flow Tables

Street Half Width = 22’
Curb Type = Rolled

| Flow Depth (ft) | Flow Area (ft) | Flooded Street Width (ft) | Flooded Parkway Width (ft) | Maximum S (ft/ft) for Y x V = 6 ($ft^2/s$) | Conveyance $Q/S^{0.5}$ (cfs) |
|---|---|---|---|---|---|
| **0.2** | 0.2 | 3.6 | 0 | 4.317 | 3.4 |
| **0.21** | 0.3 | 4.2 | 0 | 3.739 | 4.2 |
| **0.22** | 0.3 | 4.8 | 0 | 3.208 | 5.1 |
| **0.23** | 0.4 | 5.3 | 0 | 2.744 | 6.1 |
| **0.24** | 0.5 | 5.9 | 0 | 2.346 | 7.4 |
| **0.25** | 0.5 | 6.5 | 0 | 2.011 | 8.9 |
| **0.26** | 0.6 | 7.1 | 0 | 1.73 | 10.5 |
| **0.27** | 0.7 | 7.7 | 0 | 1.494 | 12.4 |
| **0.28** | 0.8 | 8.3 | 0 | 1.296 | 14.5 |
| **0.29** | 0.9 | 8.9 | 0 | 1.129 | 16.9 |
| **0.3** | 1 | 9.5 | 0 | 0.988 | 19.5 |
| **0.31** | 1.1 | 10 | 0 | 0.868 | 22.3 |
| **0.32** | 1.2 | 10.6 | 0 | 0.767 | 25.5 |
| **0.33** | 1.3 | 11.2 | 0 | 0.679 | 28.9 |
| **0.34** | 1.4 | 11.8 | 0 | 0.604 | 32.6 |
| **0.35** | 1.6 | 12.4 | 0 | 0.54 | 36.6 |
| **0.36** | 1.7 | 13 | 0 | 0.484 | 41 |
| **0.37** | 1.9 | 13.6 | 0 | 0.435 | 45.6 |
| **0.38** | 2 | 14.2 | 0 | 0.392 | 50.6 |
| **0.39** | 2.2 | 14.7 | 0 | 0.355 | 56 |
| **0.4** | 2.3 | 15.3 | 0 | 0.322 | 61.6 |
| **0.41** | 2.5 | 15.9 | 0 | 0.293 | 67.7 |
| **0.42** | 2.7 | 16.5 | 0 | 0.267 | 74.1 |
| **0.43** | 2.9 | 17.1 | 0 | 0.244 | 80.9 |
| **0.44** | 3.1 | 17.7 | 0 | 0.224 | 88 |
| **0.45** | 3.2 | 18.3 | 0 | 0.205 | 95.6 |
| **0.46** | 3.5 | 18.9 | 0 | 0.189 | 103.6 |
| **0.47** | 3.7 | 19.5 | 0 | 0.174 | 112 |
| **0.48** | 3.9 | 20 | 0 | 0.161 | 120.8 |
| **0.49** | 4.1 | 20.6 | 0 | 0.149 | 130 |
| **0.5** | 4.3 | 21.2 | 0 | 0.138 | 139.7 |
| **EXCEEDS TOP OF CURB** | | | | | |
| **0.51** | 4.6 | 21.8 | 0.5 | 0.138 | 144.2 |
| **EXCEEDS CROWN** | | | | | |
| **0.52** | 9.6 | 44 | 1 | 0.134 | 302.7 |
| **0.53** | 10.1 | 44 | 1.4 | 0.129 | 319 |
| **0.54** | 10.6 | 44 | 1.9 | 0.124 | 335.7 |
| **0.55** | 11.2 | 44 | 2.4 | 0.119 | 352.9 |
| **0.56** | 11.7 | 44 | 2.9 | 0.114 | 370.7 |
| **0.57** | 12.2 | 44 | 3.4 | 0.11 | 389 |
| **0.58** | 12.8 | 44 | 3.8 | 0.105 | 407.8 |
| **0.59** | 13.3 | 44 | 4.3 | 0.101 | 427.1 |
| **0.6** | 13.9 | 44 | 4.8 | 0.097 | 446.9 |
| **0.61** | 14.5 | 44 | 5.3 | 0.093 | 467.2 |
| **0.62** | 15.1 | 44 | 5.8 | 0.089 | 488.1 |
| **EXCEEDS R/W** | | | | | |

# Appendix B Submittal Requirements

For submittal requirements please visit the OCPW Development Services website to download the most recent information.

OC Development Services Process Checklists:

(https://ocds.ocpublicworks.com/)

1) Storm Drain Plan Review Checklist

2) Street Plan Review Checklist

3) Subdivision, Street and Local Storm Drain Improvement Plan Checklist

4) Hydrology and Hydraulics Review Checklist

5) Improvement Plan General Notes

6) Master Plan and Local Storm Drain General Notes

# Appendix C Basin Inspection Checklist

## RETARDING BASIN FACILITY INSPECTION SUMMARY

| **Facility:** | | **Inspector:** | | **Date:** |
|---|---|---|---|---|
| TYPE OF INSPECTION | Pre-Storm Season | Storm Season | Post-Storm Season | Other: |
| WEATHER | Rainy | Dry | | |
| WATER LEVEL | Dry Ponded | W.S. Elevation: | | |
| SEDIMENT LEVEL | Empty | Sediment Stored | Sediment Elevation[1]: | |

1 Sediment elevation is measured at the face of the dam embankment.

OBSERVED DEFICIENCIES, DAMAGE, OR OTHER ISSUES:

| Facility Elements:<br>*Element No.* | | *Okay* | *Exception* |
|---|---|---|---|
| 1 | Concrete "V" Ditches, Terrace and Down Drains | | |
| 2 | Basin Drainage Inlets | | |
| 3 | Sediment and Debris Basin, Debris Poles | | |
| 4 | Outlet Structures | | |
| 5 | Riprap Inlet Structures | | |
| 6 | Riprap and Erosion Control Blankets | | |
| 7 | Riprap Trapezoidal Channel | | |
| 8 | Concrete Rectangular Channel Outlet | | |
| 9 | Slotted Concrete Intake Tower | | |
| 10 | Spillway Approach Channel and Ogee Structure | | |
| 11 | Spillway Box Culvert | | |
| 12 | Energy Dissipater Structure and Baffle Blocks | | |
| 13 | Low-Level Outlet Conduits | | |
| 14 | Secondary Outlet Gate Operation | | |
| 15 | Access Roads | | |
| 16 | Dam Embankments and Graded Earth Slopes | | |
| 17 | Fencing and Gates | | |
| 18 | Reservoir Vegetation Conditions | | |
| 19 | Rodent Activity | | |
| 20 | Trash and Debris Conditions | | |
| Comments: | | | |

Inspector's Signature: ______________________________

# Appendix D Structural Design

# D.1 Transition Widths

American Concrete Pipe Association provides transition widths for pipe sizes ranging from 12-inch to 144-inch for reinforced concrete pipe in the ACPA Concrete Pipe Design Manual. A portion of the 21-inch pipe table (ACPA, 2011, Table 16) is provided herein as an example.

| TRANSITION WIDTHS IN FEET FOR 21 INCH RCP | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $K_u'$ = 0.165 | | | | Ku' = 0.150 | | | | $K_u'$ = 0.130 | | | | $K_u'$ = 0.110 | | | |
| Cover (ft) | Type 1 | Type 2 | Type 3 | Type 4 | Type 1 | Type 2 | Type 3 | Type 4 | Type 1 | Type 2 | Type 3 | Type 4 | Type 1 | Type 2 | Type 3 | Type 4 |
| 5 | 3.8 | 3.9 | 3.9 | 4.0 | 3.7 | 3.8 | 3.8 | 4.0 | 3.6 | 3.8 | 3.8 | 3.9 | 3.6 | 3.7 | 3.7 | 3.8 |
| 6 | 3.9 | 4.0 | 4.0 | 4.1 | 3.8 | 3.9 | 3.9 | 4.1 | 3.7 | 3.9 | 3.9 | 4.0 | 3.6 | 3.8 | 3.8 | 3.9 |
| 7 | 4.0 | 4.1 | 4.1 | 4.2 | 3.9 | 4.1 | 4.1 | 4.2 | 3.8 | 3.9 | 3.9 | 4.1 | 3.7 | 3.8 | 3.8 | 4.0 |
| 8 | 4.1 | 4.2 | 4.2 | 4.4 | 4.0 | 4.2 | 4.2 | 4.3 | 3.9 | 4.0 | 4.0 | 4.2 | 3.8 | 3.9 | 3.9 | 4.0 |
| 9 | 4.2 | 4.4 | 4.4 | 4.5 | 4.1 | 4.3 | 4.3 | 4.4 | 4.0 | 4.1 | 4.1 | 4.3 | 3.9 | 4.0 | 4.0 | 4.1 |
| 10 | 4.3 | 4.5 | 4.5 | 4.6 | 4.2 | 4.4 | 4.4 | 4.5 | 4.1 | 4.2 | 4.2 | 4.3 | 4.0 | 4.1 | 4.1 | 4.2 |

**Appendix Table D-1: Transition Width in Feet for 21 Inch RCP Excerpt (ACPA, 2011, Table 16)**

## *D.2 Settlement Design Values of Ratio and Coefficient of Cohesion*

| Installation and Foundation Condition | Settlement Ratio $r_{sd}$ | |
|---|---|---|
| | Usual Range | Design Value |
| **Positive projecting** | **0.0 to + 1.0** | |
| **Rock or unyielding Soil** | **+1.0** | **+1.0** |
| ***Ordinary soil** | **+0.5 to 0.8** | **+0.7** |
| **Yielding Soil** | **0.0 to +0.5** | **+0.3** |
| **Zero Projecting** | | **0.0** |
| **Negative projecting** | **-1.0 to 0.0** | |
| **p' = 0.5** | | **-0.1** |
| **p' = 1.0** | | **-0.3** |
| **p' = 1.5** | | **-0.5** |
| **p' = 2.0** | | **-1.0** |

*The value of the settlement ratio depends on the degree of compaction of the fill material adjacent to the sides of the pipe. With good construction methods resulting in proper compaction of bedding and side fill materials, a settlement ratio design value of +0.5 is recommended.

**Appendix Table D-2: Settlement Design Values of Ratio and Coefficient of Cohesion (ACPA Design Manual, Table 40)**

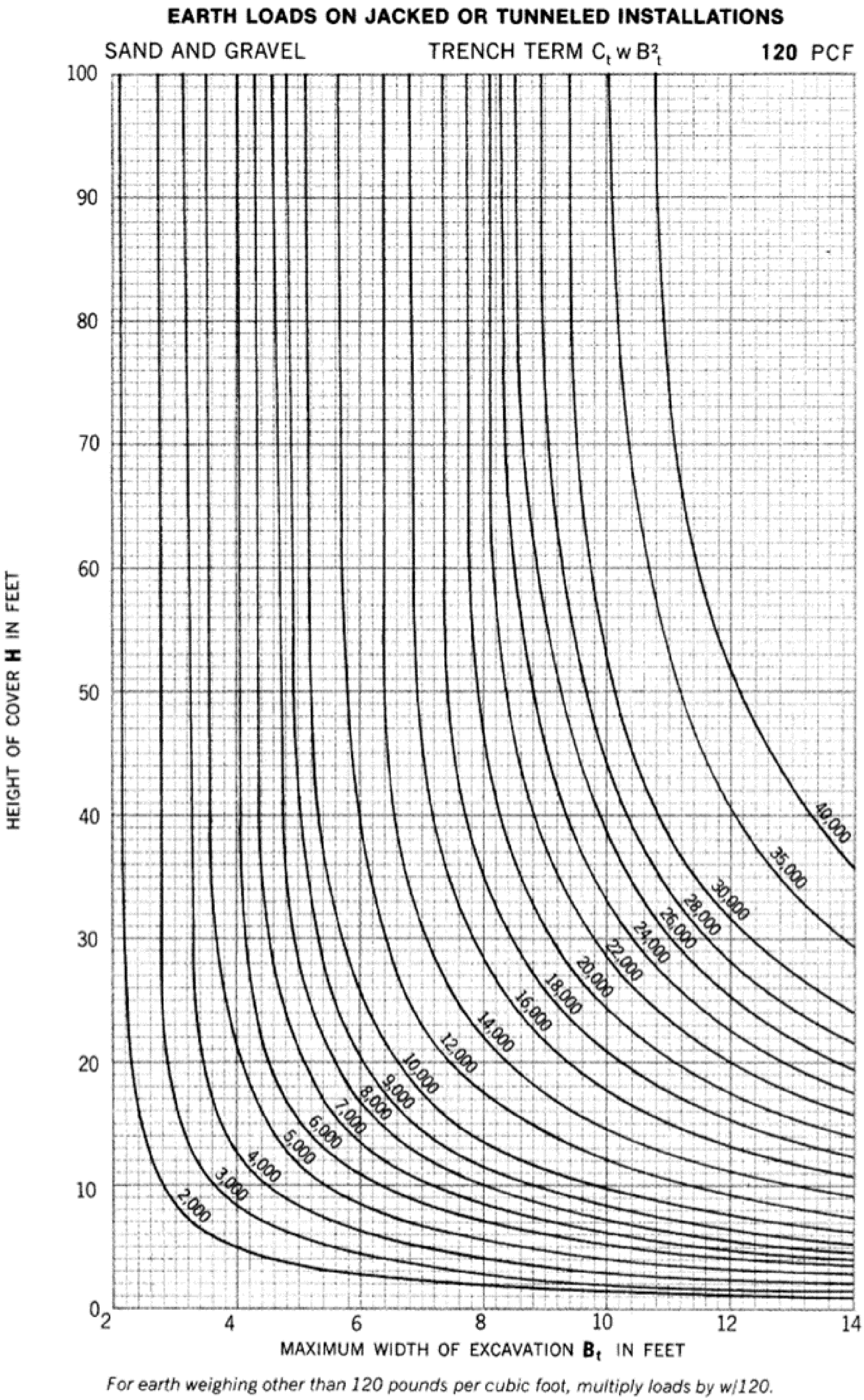
| Type of soil | Values of c (psf) |
|---|---|
| Clay | |
| • Soft…………………… | 40 |
| • Medium…………….. | 250 |
| • Hard………………….. | 1,000 |
| Sand | |
| • Loose Dry…………….. | 0 |
| • Silty……………………. | 100 |
| • Dense………………….. | 300 |
| Top Soil | |
| • Saturated………………….. | 100 |

**Appendix Table D-3: Coefficient of Cohesion (ACPA Design Manual, Table 41)**

## D.3 Earth Loads on Jacked or Tunneled Installations

The American Concrete Pipe Association (ACPA) Concrete Pipe Design Manual (ACPA, 2011) gives Figures 147, 149, 151, and 153 with "trench term $C_t w B^2_t$" for the various soil conditions. ACPA provides figures for all the soil bedding. Figure 147 (Sand and Gravel) follows as an example.

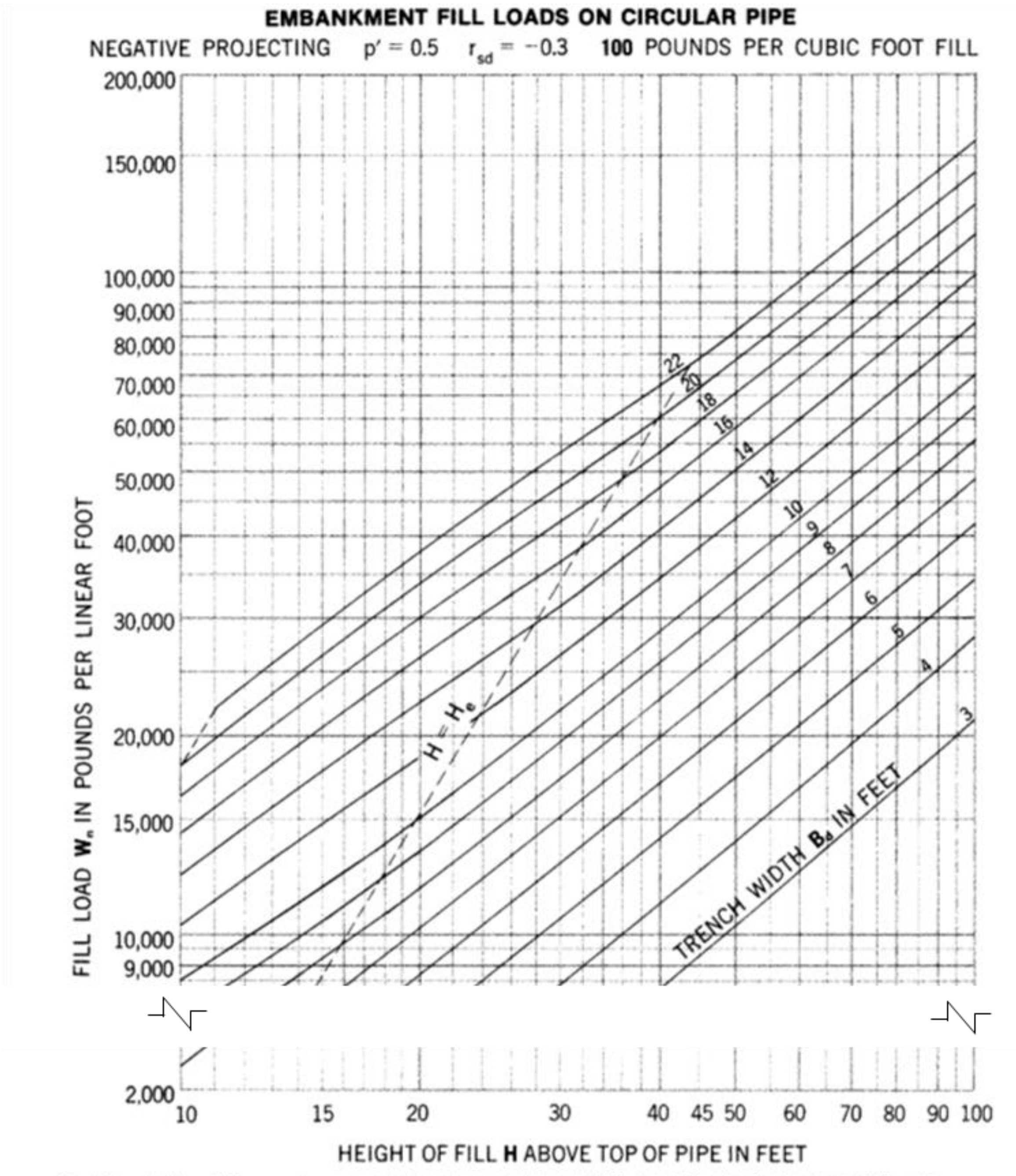
Figures 148, 150, and 152 in the Concrete Pipe Design Manual (ACPA, 2011) give values of the cohesion term, $2cC_tB_t$, divided by the design value for the coefficient of cohesion (c). To obtain the total earth load for any given height of cover, with of bore or tunnel and type of soil, the value of the cohesion term must be multiplied by the appropriate coefficient of cohesion (c). Then this product is subtracted from the value of the trench load.




**Appendix Figure D-1: Earth loads and maximum excavation widths (ACPA, 2011, Figure 147)**

## D.4 Negative Projecting Embankment Fill Loads

Negative projecting fill loads on circular pipe can be found on Figure 195 to 213 of the American Concrete Pipe Association Concrete Pipe Design Manual. A portion of Figure 196 is provided herein.




*For fill weighing 110 pounds per cubic foot, increase loads 10%; for 120 pounds increase 20%, etc. Interpolate for intermediate trench widths.*

**Appendix Figure D-2: Embankment Fill Loads on Circular Pipe (ACPA, 2011, Figure 196)**

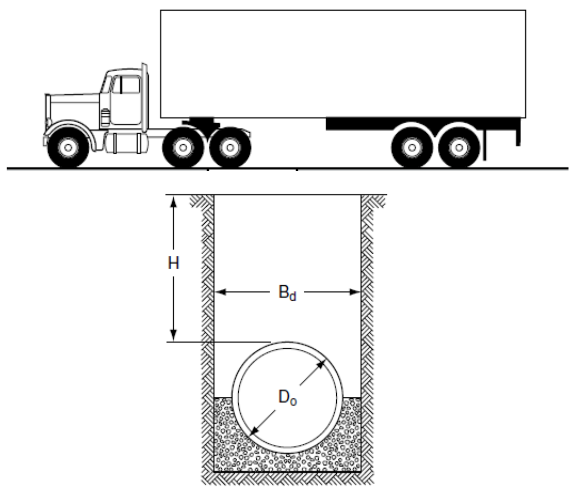
## D.5 Example

**Given:**

A Wall B type pipe that is 30 in diameter is to be installed as a storm drain subjected to AASHTO HL-93 highway loadings. The pipe will be installed in a trench per OCPW Standard Plan 1319 with a minimum of 30 in of cover over the top of the pipe. The AASHTO LRFD criteria will be used with Select Granular Soil and Type 2 installation (OCPW standard 1319). Typical single truck case with wheel spacing of 6 ft is to be considered.

Unit weight of soil, $\gamma_s = 120$ (lb/ft$^3$)

Height of cover, $H$ =30-inch




**Appendix Figure D-3: 5-Axle Truck with 6 ft Spacing**

**Find:**

1. The trench width per OCPW Standards
2. The maximum 0.01 inch-crack D-load (with safety factor =1.25) required of the pipe.

**Solution:**

<u>Determine the Trench Width</u>

The trench width, B, is to be provided by the Designer. OCPW requires the use of Standard Plan 1319. Therefore, the recommend installation trench width for a 30-inch pipe would be:

$$B_d = D_O + 2W$$

Where W is the distance from the outside edge of the pipe to the edge of the trench wall per Standard Plan 1319

The wall thickness for a 30-inch Wall B pipe is 3.5-inch (see Table 10-7). The outside pipe diameter is:

$$D_0 = \frac{30+2(3.5)}{12}$$

$$D_O = 3.08\ ft$$

Per Standard Plan 1319: if H ≤ 8 ft, then W ≥ 6 in

$$B_d \geq 3.08 + 2 * 0.5 = 4.08\ ft$$

<u>Determine the Earth Load ($W_E$)</u>

Check the trench transition width.

ACPA provides tables with transition widths, where once exceeded, the trench condition transitions to an embankment condition. Appendix D-1 provides a sample of the tables from the ACPA Concrete Pipe Design Manual. By using these tables, it can be determined that the 4.08 ft wide trench exceeds the transition width due to inadequate fill height. Therefore, the earth load must be evaluated as a positive projecting condition.

For a positive projection embankment condition:

$$W_E = PL * VAF$$

$$PL = \gamma_s [H + \frac{D_O(4-\pi)}{8}] D_O$$

$$PL = 120\left[2.5 + \frac{3.08(4-\pi)}{8}\right] 3.08$$

$$PL = 1046\ lb/ft$$

Table 10-5 gives VAF for the 3 standard installation types. Use VAF of 1.40 for type 2 installation (OCPW Standard Plan 1319) to calculate the earth load.

$$W_E = 1046 * 1.40$$

$$W_E = 1465\ lb/ft$$

<u>Determine the Fluid Load ($W_F$)</u>

$$W_F = \gamma_w \times A$$

$$W_F = 62.4 \times \frac{\pi(2.5)^2}{4}$$

$$W_F = 307 \text{ (lb/ft)}$$

Calculate Total Live Load ($W_T$)

Table 42 in the Concrete Pipe Design Manual (ACPA, 2011) can be used to look up the live load for various pipe sizes and fill depths. The live load can also be calculated directly.

Height of earth cover is 2.5 feet. Use Standard Plan 1319 and AASHTO LRFD with select granular soil fill. From Illustration 4.12 (ACPA, 2011) the critical load, is 32,000 pounds from a single dual wheel traveling perpendicular to the pipe length.

Using Illustration 4.12 (ACPA, 2011)

2.03 ft < H < 5.5 ft therefore P = 32,000 lb

Calculate the Average Pressure Intensity (ACPA, 2011)

$$w = \frac{P(1 + IM)}{A}$$

The spread area is:

A = (Spread A) (Spread B)

For H = 2.5 ft

Spread A = 1.67 + 4 + 1.15H = 8.5 ft

Spread B = 0.83 + 1.15H = 3.6 ft

A = 8.5 × 3.7 = 31.7 $ft^2$

Impact Factor (ACPA, 2011):

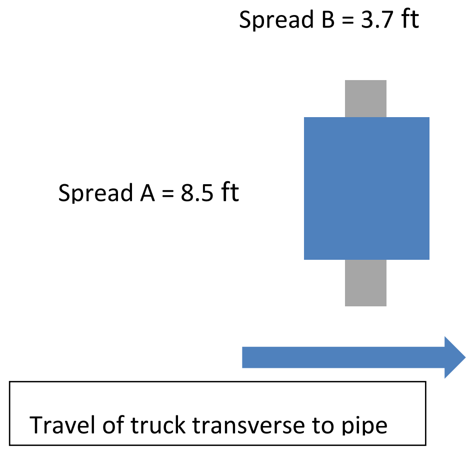
$$IM = \frac{33(1 - 0.125H)}{100} = \frac{33(1 - 0.125 * 2.5)}{100} = 0.23$$

$$w = \frac{32000(1 + 0.23)}{31.6} = 1245 \text{ psf}$$

Calculate total live load acting on the pipe (ACPA, 2011).

$$W_T = (w + L_L)L \times S_L$$

Assuming truck travel transverse to pipe centerline



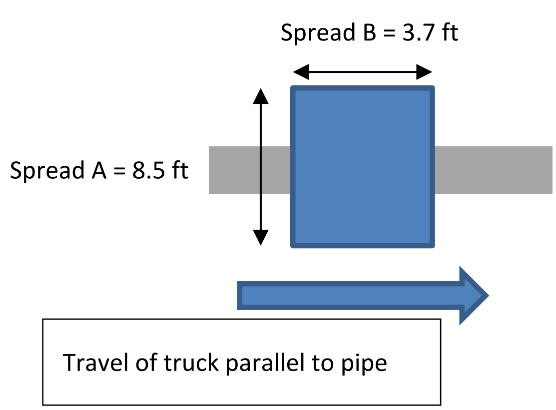

$L_L$= 64 psf (based on H magnitude, refer to ACPA, 2011)

L = Spread A = 8.5 ft

$B_C = D_O$ = 3.08 feet, which is less than spread B, therefore $S_L$ = 3.08 feet

$W_T = (1245 + 64) \times 8.5 \times 3.08 = 34{,}270$ lb

Assuming truck travel parallel to pipe center line




$L_L$ = 64 psf

L = Spread B = 3.6 ft

$B_C$= 3.08 ft, which is less than Spread A, therefore $S_L$ = 3.08 ft

$W_T = (1{,}245 + 64) \times 3.6 \text{ x } 3.08 = 14{,}514$ pounds

Use $W_T$ = 34,270 lb. for truck travel perpendicular to pipe center line.

Calculate live load on pipe in pounds per lineal foot, ($W_L$) (ACPA, 2011)

$$W_L = \frac{W_T}{L_e}$$

$$L_e = L + 1.75(\frac{3}{4R_O})$$

$R_o = D_O$ = 3.08 feet

$$L_e = 8.5 + 1.75\left(\frac{3}{4 * 3.08}\right) = 8.9\ ft$$

$$W_L = \frac{34{,}270}{8.9} = 3850\ lb/ft$$

Determination of Bedding Factor, $B_f$

- Determine the Embankment Bedding Factor, $B_{fe}$, (refer to ACPA, 2011 for types of bedding factors).
- The embankment bedding factor for a type 2 installation may be interpolated from Appendix Figure D-4.

| | **Standard Installation** | | |
|---|---|---|---|
| **Pipe Diameter** | Type 1 | Type 2 | Type 3 |
| **12 inch** | 4.4 | 3.2 | 2.5 |
| **24 inch** | 4.2 | 3.0 | 2.4 |
| **36 inch** | 4.0 | 2.9 | 2.3 |
| **72 inch** | 3.8 | 2.8 | 2.2 |
| **144 inch** | 3.6 | 2.8 | 2.2 |

**Appendix Figure D-4: Embankment Bedding Factors (ACPA, 2011)**

$B_{fe24}$ = 3.0
$B_{fe36}$ = 2.9
$B_{fe30}$ = ((30 – 36)/ (24-36)) * (3.0 – 2.9) +2.9 (linearly interpolate)
$B_{fe30}$ = 2.95

<u>Determination of Live Load Bedding Factor</u>

The live load bedding factor for a 30-inch pipe under 30 inches of cover can be read from Appendix Table D-4 (or linearly interpolated when necessary). Refer to the Concrete Pipe Design Manual (ACPA, 2011) for application of live load bedding factors.

$B_{fLL} = 2.2$

| Fill Height in Feet | 12" Pipe | 24" Pipe | 36" Pipe | 48" Pipe | 60" Pipe | 72" Pipe | 84" Pipe | 96" Pipe | 108" Pipe | 120" Pipe | 144" Pipe |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.5 | * | * | * | * | * | * | * | * | * | * | * |
| 1.0 | 2.2 | 2.2 | 1.7 | 1.5 | 1.4 | 1.3 | 1.3 | 1.3 | 1.1 | 1.1 | 1.1 |
| 1.5 | 2.2 | 2.2 | 2.1 | 1.8 | 1.5 | 1.4 | 1.4 | 1.3 | 1.3 | 1.3 | 1.1 |
| 2.0 | 2.2 | 2.2 | 2.2 | 2.0 | 1.8 | 1.5 | 1.5 | 1.4 | 1.4 | 1.3 | 1.3 |
| 2.5 | 2.2 | 2.2 | 2.2 | 2.2 | 2.0 | 1.8 | 1.7 | 1.5 | 1.4 | 1.4 | 1.3 |
| 3.0 | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 1.8 | 1.7 | 1.5 | 1.5 | 1.4 |
| 3.5 | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 1.9 | 1.8 | 1.7 | 1.5 | 1.4 |
| 4.0 | * | * | * | * | * | * | * | * | * | * | * |
| 4.5 | * | * | * | * | * | * | * | * | * | * | * |
| 5.0 | * | * | * | * | * | * | * | * | * | * | * |

*Intentionally blanked, see ACPA, 2011

**Appendix Table D-4: Live Load Bedding Factor (ACPA, 2011)**

<u>Application of the Factor of Safety</u>

A Factor of Safety (FS) of 1.25 will be used by Orange County.

Determination of Pipe Strength required

$W_E$ = 1,465 lb/ft

$W_F$ = 307 lb/ft

$W_L$ = 3,850 lb/ft

$B_f$ = $B_{fe}$ = 2.95

$B_{fLL}$ = 2.2

D = 30 in (inside diameter of pipe)

The D-load for the 0.01-inch crack is (ACPA, 2011):

$$\text{D load} = \left(\frac{W_E + W_F}{B_f} + \frac{W_L}{B_{fLL}}\right) \times FS \times \frac{12}{D}$$

$$\text{D load} = \left(\frac{1465 + 307}{2.95} + \frac{3850}{2.2}\right) \times 1.25 \times \frac{12}{30}$$

$D_{0.01}$ = 1175 psf

**A trench with a minimum width of 4.08 ft should be used per OCPW standards.**

A 30-inch diameter pipe with Wall B thickness that can withstand a minimum three-edge bearing test (for the 0.01 inch crack) of 1175 pounds per linear foot per foot of inside diameter would be required (with a Factor of Safety =1.25). It is common practice to round up D-loads to the nearest 50. The D-load on the Plan in this case would be specified as 1200-D. A class III pipe which is manufactured to withstand up to 1350-D for formation of the 0.01-inch crack would be expected. However, the D-load tables provided in the Orange County Local Drainage Manual are intended to be more conservative, especially at shallow depths. D-loads in the tables have been adjusted to higher values for conservative design. **The D-load tables indicate a minimum D-load of 1750-D.**

# Appendix E 1996 LDM Izzard Inlet Method

**Previous 1996 OCLDM Manual's CurbOpening Inlet Formula (Izzard 1950)**

1. Capacity of Curb Opening Inlets with Partial Interception

1. General

For the "continuous grade" condition, the capacity of the inlet is dependent upon many factors including gutter slope, depth of flow in the gutter, heights and length of curb opening, street cross-slope, and the amount of depression at the catch basin. In addition, all of the gutter flow will generally not be intercepted and some flow will continue past the inlet area ("inlet carryover"). The amount of carryover must be included in the drainage facility evaluation as well as in the design of the inlet.

2. Depth of Water at Inlet

The depth of the water at the inlet entrance for a given discharge varies directly with:

(a) Cross slope of the pavement at the curb. ($S_x$)
(b) Amount of warping or depression of the gutter flow line at the inlet.
(c) Roughness of the flow line. ("n")
(d) Longitudinal slope of the gutter. (S)

3. Capacity of Standard Inlet

The capacity (Q) of a curb opening (standard curved face plate inlet) when intercepting 100 percent of the flow in the gutter is given by the formula:

$$Q = 0.7\ L\ (a_d + y)^{3/2}$$

Where y = depth of flow in approaching gutter

$a_d$ = depth of depression of curb at inlet

L = length of clear opening

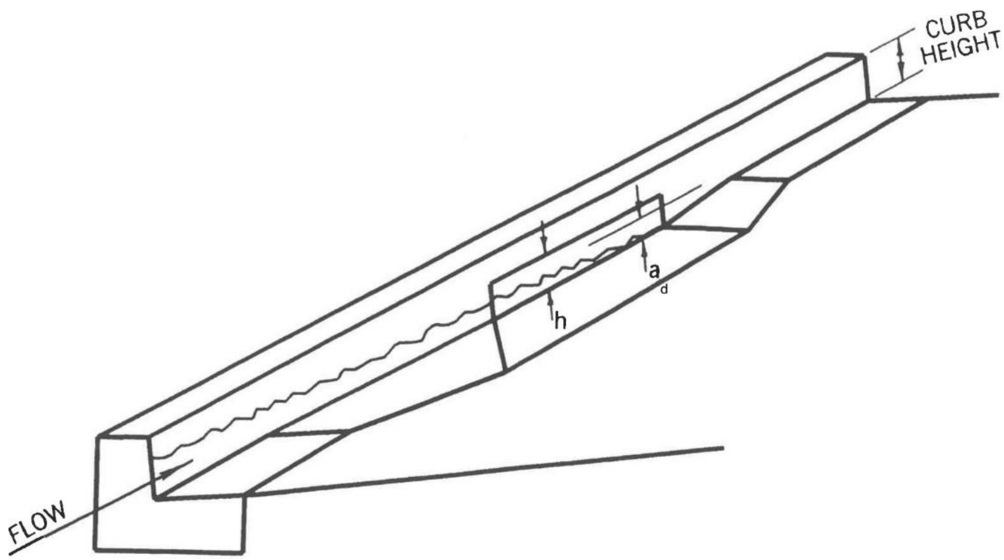
4. Sizing Length of Inlet

To size an opening length the following information must be known:

(a) Height of the curb opening (h).
(b) Depth ($a_d$) of flow-line depression, if any, at the inlet.
(c) Design discharge (Q) in the gutter (drainage area, rainfall intensity and runoff coefficients are included in the hydrology design discharge analysis). Any carryover from a previous inlet must be included.
(d) Depth of flow in normal gutter for the particular longitudinal and cross-slopes at the inlet in question. This may be determined from the street capacity charts.

2. <u>Design Procedure for Continuous Grade</u>

The capacity and length of a curb opening inlet may be decreased by allowing part of flow to pass the opening. A maximum of fifteen percent is recommended to be bypassed.




**Appendix Figure E-1: Definition Figure for Inlets (refer to chapter 3)**

Appendix A Determine Q to inlet.

Appendix B Determine depth of flows (y) in street using street capacity tables (Appendix A).

Appendix C Determine best design of inlet to use, checking that depth of depression at curb inlet plus depth of flow in approach gutter ($a_d$ + y) is less than the height of the curb opening per (Table 3-5).

Appendix D Enter Appendix Figure E-2(A) with flow depth, y, and gutter depression at the inlet, "$a_d$", and determine Q/L the interception per foot of inlet opening if the inlet were intercepting 100% of the gutter flow.

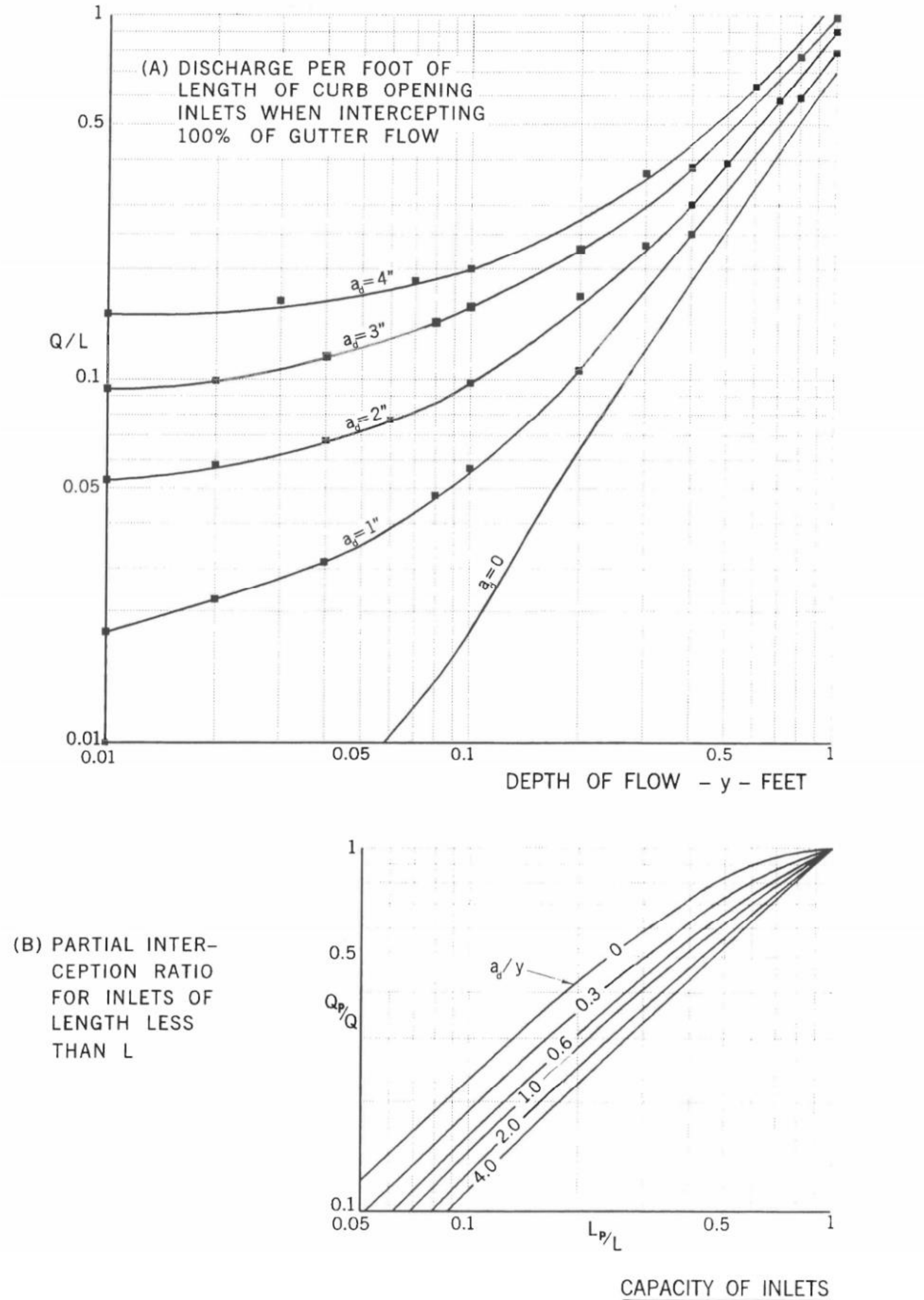
Appendix E Determine length of inlet L required to intercept 100% of the gutter flow. L = total gutter flow Q divided by the factor Q/L. If single inlet is to be used to intercept the flow, choose inlet length greater than or equal to L. If multiple inlets are to be used, go on to #6.

Appendix F Compute ratio Lp/L where Lp = actual length of inlet for partial interception.

Appendix G Enter Appendix Figure E-2(B) with Lp/L and $a_d$/y and determine ratio Qp/Q, the proportion of the total gutter flow intercepted by the inlet in question shall not be less than 0.7.

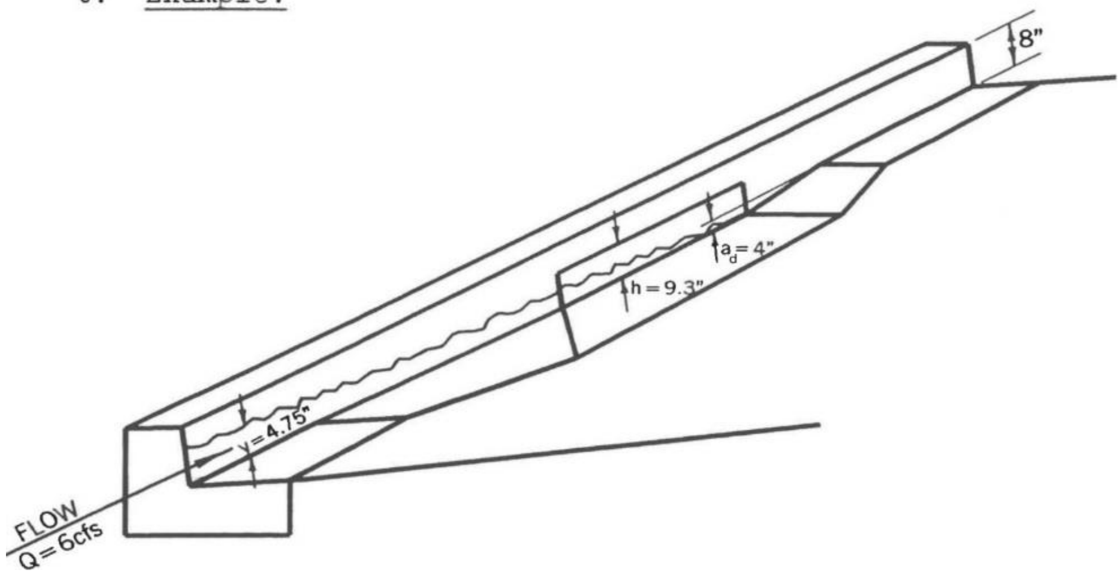
Appendix H The partial flow intercepted Qp, is the ratio Qp/Q times the total gutter flow Q.

Appendix I The flow carried over in the street to next inlet Qc, is then Q - Qp, return to #1.




**Appendix Figure E-2: Capacity of Inlets**

6. Example:




**Appendix Figure E-3: Example Definition**

- Given:

  Q in street = 6 cfs
  Curb height = 8″
  Depth of water at curb, y = 4.75″
  Height of local depression, ad = 4″

  Use standard curb face (see Figure 3-6)

- Calculations:

  Height of curb opening, h = 9.3″ (see Figure 3-13 and Table 3-5 in Chapter 3)

  Check that $h > a_d + y$

  9.3″ > 4.75" + 4″ (OK)

  from Appendix Figure E-2 (A) (for $a_d$ =4″, y= 4.75″ = .4′)

  Q/L = 0.44 cfs/ft

  Therefore, length of catch basin

  L = Q/(Q/L) = 6/0.44 = 13.6 ft

  Use 14 ft catch basin.

- Standard Calculation Format

  A standard form calculation format has been included as Appendix Figure E-4.

Page ____ of ______

Project: ______________________________

Designer: ______________________________

Location/Street: ______________________________

Date: ______________________________

Inlet # ____________________________

C U R B O P E N I N G (I n t e r c e p t i o n)

Plan Sketch

GIVEN:

(a) Discharge Q ___ = ______ CFS

(b) Street slope S = ______ '/'

(c) Curb type "A2-____" "D" other ______

(d) Half street width = ______ ft.

SOLUTION: Street capacity table reference ________________________

$Q/S^{1/2}$ = ______/(______)$^{1/2}$ = ______. Therefore y = ______.

Q/L = ______ [from Figure 5-10(A)]

L = Q/(Q/L) = ______/______ = ______ (L for total interception)

TRY:

Lp = ______ ft.

Lp/L = ______/______ = ______

$a_o/y$ = .33/______ = ______

$Q_p/Q$ = ______ [from Figure 5-10(B)]

$Q_p = (Q_p/Q) \times Q$ = ______ X ______ = ______ CFS (Intercepted)

$Q_c = Q - Q_p$ = ______ - ______ = ______ CFS (Carryover)

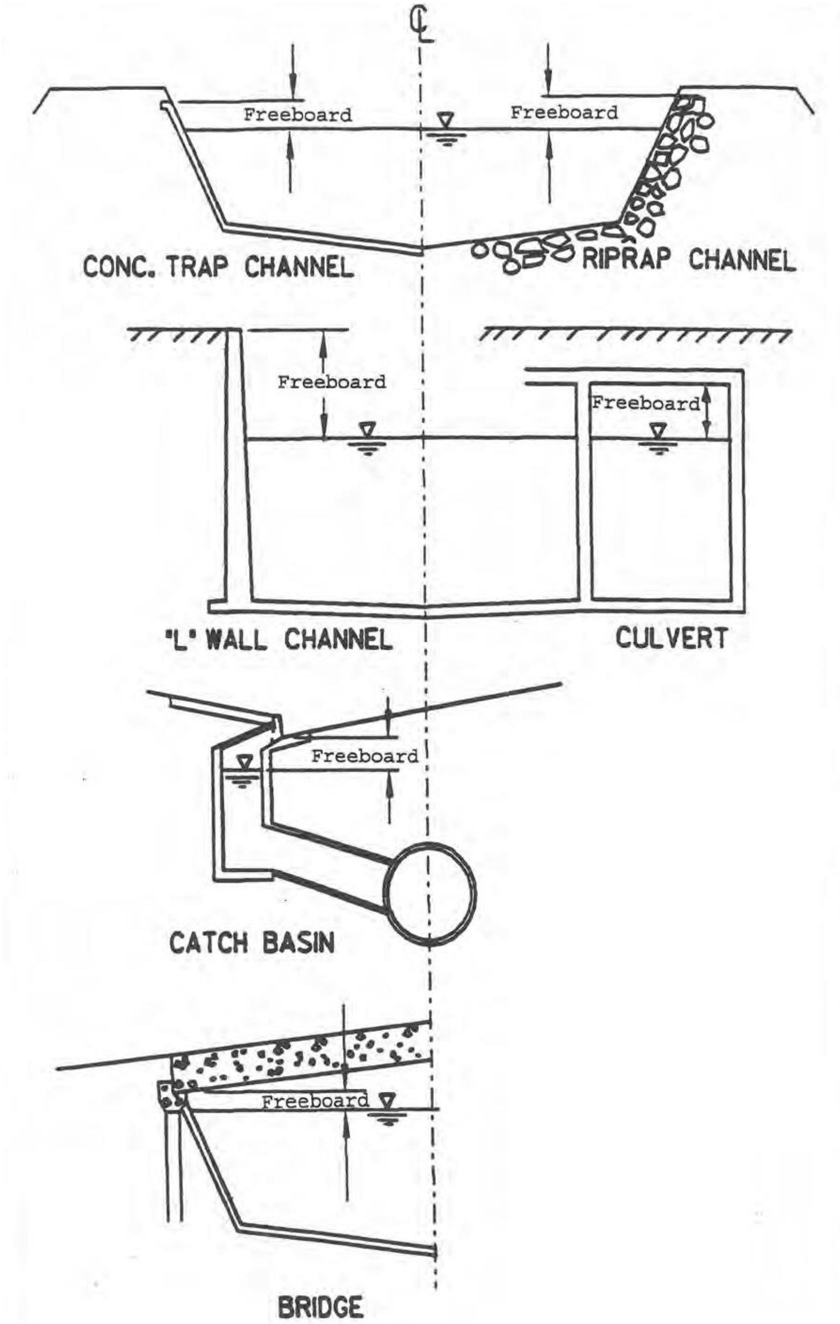
Curb Opening Calculation Form

**Appendix Figure E-4: Curb Opening Calculation Form**

# Appendix F Freeboard

Freeboard is the vertical distance from the design hydraulic grade line as defined below and as shown in the following figure.

1. Top of levee in ultimate unlined earth levee channels
2. Top of rock where riprap slope protection is utilized
3. Top of wall or structural section in concrete channels
4. Soffit where box conduits or culverts are designed as open channels
5. Low Point of the soffit of bridges
6. Where necessary, add wave height, superelevation, and/or any other factors required to be separately evaluated




**Appendix Figure F-1: Freeboard Definitions**